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UNITED STATES
DEPARTMENT OF THE INTERIOR
Bureau of Reclamation

WATER SYSTEMS OPERATION AND MAINTENANCE WORKSHOP

1990

SESSION NOTES

ENGINEERING DIVISION
DENVER OFFICE
DENVER, COLORADO

I 27.2:

As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering the wisest use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to assure that their development is in the best interests of all our people. The Department also has a major responsibility for American Indian reservation communities and for people who live in Island Territories under U.S. Administration.

Mission of the Bureau of Reclamation

The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.

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Water Systems Operation and Maintenance Workshop - 1990

MAINTENANCE MANAGEMENT

Session 1-1

Darrel Krause and Bill Bouley

Maintenance is the upkeep of facilities with goals of efficient operation, minimum breakdowns, good appearance, reasonable costs, extended useful life, and safety.

Some of the results of a good maintenance management or preventive maintenance (PM) program are:

. better and more timely deliveries
. less breakdown maintenance
. fewer emergencies
. better budgeting and planning of work
. conservation of water
. improved water quality
. better overall efficiency
. minimizing shortages
. getting "on top" of run down system
. fewer complaints
. fewer accidents
. allowing easier setting of standards

A good program requires that problems or potential problems be identified at an early stage. This identification process is critical to the success of a good maintenance program in your organization.

Maintenance programs may take different forms and should be adapted to best fit your organization. A suggested procedure for development of a good maintenance program is given below Operational aspects should also be considered in this process and ultimately O&M requirements should lead to the development of annual (or biannual, triannual) work plans and budgets. The program may consist of the following:

1. Maintenance Management System

2. Annual reviews by irrigation district or other operating entity personnel

3. Periodic review (RO&M -- Review of Operation and Maintenance Program)

The implementation of these three aspects are discussed in detail below:

1. Maintenance Management System (MMS) - The procedures and details for establishment and implementation of an MMS are given in "Guidelines for Manual Maintenance Management System for Water O&M Projects" which are attached as part of these notes.

2. Annual Reviews by Irrigation District Personnel - Annual onsite inspections and meetings should be held for the purposes of identifying deficiencies in the operation and maintenance program and planning and prioritizing future work. These reviews should be attended by managers, supervisory personnel, directors, water users and their representatives, and O&M personnel directly involved in the operation of the system.

Tours should be scheduled to examine system deficiencies. The success of the tour depends on how well it is planned and attended. Transportation should be provided which allows the entire group to travel together so that discussions can be held while traveling through the project.

If tours cannot be conducted, photographs and slides can be used to show field accomplishments and needs. Both the tours and photographs should point out current problems, accomplishments, and suggested future improvements. Agreement on the work required and schedules should be reached as a result of these reviews.

These reviews, if properly conducted, identify deficiencies and can be beneficial in the development of work plans and budgets. They can also help in establishing a good communication process among all involved O&M and administrative personnel.

3. Periodic Review (RO&M Program) - Periodic oversight reviews should be held to evaluate system condition and to provide assurance that proper operation is sustained on a long term basis to fulfill intended objectives of the project.

These reviews could be similar to the USBR Review of Operation and Maintenance examinations which are held every 3 years by Regional Office and Denver Office personnel. The teams performing the examinations should be experienced in operation and maintenance of irrigation systems and system evaluation and they should not be directly involved in project operations. The reviews should include representative portions of distribution systems and all dams as well as equipment and operational and management aspects. A report on each review should be prepared and copies distributed to appropriate office heads and water user organizations. The report should list recommendations that require corrective actions.

A more detailed discussion of this program is discussed in "Review of Facilities" which is attached.

Summary - A good, well organized O&M program is critical to sustained successful operation of water systems (distribution systems and dams). The method described above is one way of accomplishing such a program. Other similar programs may be more adaptable to your system and they should be considered when implementing O&M programs. The basic premise is that potential problems should be identified and corrected before they happen and create emergencies, safety problems, and general system deterioration.

PAY ME NOW OR PAY ME LATER!

Guidelines

for

Manual Maintenance Management System (MMS)

For Water O&M Projects

Foreword

The Commissioner of Reclamation, by memorandum of January 18, 1984, directed all regional and project offices to initiate implementation of POMMS in 1984. A task group was established to evaluate the program for use on irrigation projects and establish guidelines to be used by regional and project offices. It was the conclusion of the task group that POMMS was too complex, and in many cases uneconomical, for use in some of the smaller, single purpose irrigation projects, and a simple system should be established that may be easily adapted to the POMMS system at a later date. As a result, it was recommended that a manual maintenance management system (MMS), such as a 52-week file with a work order system, be developed for use on irrigation projects. A sample MMS has been developed and tested by the task group and is included as part of these guidelines.

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Foreword

1. General
2. Preventive Maintenance
3. Safety
4. Work Schedules
5. Inspection and Preventive Maintenance Check List
6. Inspection and Maintenance Record Cards
7. Records of Inspections, Maintenance, and Tests
8. Special Work Orders

1. General - The apparatus, structures, mechanical equipment, etc., which are parts of water storage, transmission, and distribution systems must be maintained in a satisfactory operating condition at all times. Following an established maintenance program will result in the best service to water users, fewer emergency repairs, better working conditions for maintenance personnel, and lower maintenance costs. Because of the great variety of equipment used on Reclamation projects, these guidelines will serve as an outline or guide for the maintenance procedures which should be carried out.

2. Preventive Maintenance - A continuous program of preventive maintenance, combined with equipment overhaul when required, will best suit the needs on Reclamation projects. This method consists of continually performing routine maintenance and adjustment work to the critical items of a structure or facility in a scheduled and organized manner so as to make special inspection and heavy repairs less necessary. Maintenance is applied more specifically to the parts which are known to need repairs or replacement most often. The underlying theory is that proper lubrication, adjustment, and small repairs, applied often enough to the critical parts, will make it less necessary to undertake complete overhaul. Experience shows that breakdowns are reduced, outages shortened, and the frequency for overhaul of most facilities is lessened when preventive maintenance is followed. Depreciation and obsolescense of equipment may often be reduced by using improved repair materials and parts which are at least equal to the original. The success of preventive maintenance will depend largely on the alertness of the personnel who work around the equipment. Continuous attention should be given to such things as loose bolts, vibration, leaks, wear, unusual noise, odor, etc.

3. Safety - All work should be conducted in a safe manner. A job plan and safety analysis should be completed for each job and reviewed with all involved personnel prior to undertaking the work (see blank form - Exhibit 1). Copies of the job plan and safety analysis should be filed in the equipment file.

4. Work Schedules - Maintenance work should be scheduled to make the best use of available personnel to coincide with favorable weather so far as possible, and to coincide with periods when outages of the equipment can best be tolerated. In following the preventive maintenance method, the work to be done is spread out over the entire year. This makes more efficient use of personnel. Maintenance work should also be divided into the group which can be done at any time and that which can be done only at times when the equipment can be taken out of service.

5. Inspection and Preventive Maintenance Check Lists - Each structure or piece of equipment requires inspection and preventive maintenance at specified intervals. In order to carry out such a program, it is necessary to follow an effective but simple method of scheduling the work and keeping records of inspections and repairs. The check lists included in these guidelines, as Exhibit 2, should be used as a guide to proper maintenance. The time intervals shown represent the average intervals which the particular item should be inspected. More or less frequent inspections may be necessary on

some items as operating experience or accessibility for maintenance dictates. Anything unusual found on such inspections should be promptly acted upon. The indexed paragraphs following the check lists are intended to give pertinent instructions on what to look for and what to do. However, it is not intended to duplicate instructions already covered in detail in other irrigation manuals, manufacturers' instruction books, or other available publications.

6. Inspection and Maintenance Record Cards - A card system should be kept for recording inspection and maintenance data and is recommended for calling attention to the time when an inspection is due and for recording the data and brief description of the work done. A 5- by 8-inch card, form SVC-140, shown as Exhibit 3, is available for this purpose.

The front of this card has spaces for recording the nameplate and other pertinent data for the specific equipment. The back of the card has spaces for recording the dates on which inspections and repair work were done, name of the inspector, a reference to the complete inspection and repair report, a brief description of the work done, and time required for work done. Another card can be used as a continuation of the original card when the latter is filled up. It should be stapled to the original card. Entries should be made on the "Inspection and Maintenance Record" for the detailed semiannual or annual inspection and repair work but not for the routine daily or weekly inspections unless some repair work is performed.

A card should be used for each piece of equipment which is subject to scheduled periodic inspection and maintenance. A rotating file system, illustrated in Exhibit 4, is suggested. This entire file, including the index tabs, rotates forward as time goes on, with cards removed from the front as work is done being placed in the back so that for each year (or other inspection interval) the cards come up in the same order. The file should be consulted at least monthly to see what work is due. After the work is completed, the card should be filled in, signed by the maintenance foreman, and placed back in the file.

A suggested file numbering system for the cards is illustrated in Exhibit 4. The first symbol consists of the equipment designation symbol (see Exhibit 5) plus the location designation, unit number, and activity code. The second symbol is the month in which the inspection and preventive maintenance work is due. For a semiannual card file, the second symbol should show two months, and the index tabs should be marked in a similar manner. This file will rotate twice in a year. If desired, a third symbol may be used to indicate the week due.

Since it may be desired at times to find a card applying to some particular piece of equipment which may be at some unknown place in the file, a separate equipment check list, see Exhibit 2, is needed. From this list, the card for any piece of equipment can be promptly located from the card file by the month in which it is filed. The list also serves as a check on file cards which may have been misplaced or lost. The checklist should also show the reference file number.

7. Records of Inspections, Maintenance, and Tests - Keeping records of the inspections, repairs, and tests is an important part of preventive maintenance. It provides a means of recognizing chronic problems and finding the causes, and provides a basis for estimating what spare parts should be kept on hand and how often repairs will be necessary. The card described in paragraph "Inspection and Maintenance Record Cards" above, when used in the rotating file system, is intended to serve primarily as an index and tickler system to call attention to when inspection should be made and with only a brief reference to what was done.

A more detailed report should be made and filed in the reference file for future reference to document extensive or unusual maintenance.

8. Special Work Orders - During the life of a project, extraordinary maintenance or equipment replacement which cannot be scheduled on a routine basis becomes necessary. When this happens, a special request must be made to accomplish this work. In order to assure that the work is performed in an expedient manner and properly documented, it is suggested that a "Work Order," form 7-2053 (shown as Exhibit 6), be used. This form was developed for use on POMMS; it is easy to use and provides the necessary information to perform the work. Columns on the form, which are applicable to POMMS, can be left blank when used for the manual system.

JOB PLAN AND SAFETY ANALYSES
RIO GRANDE PROJECT

Exhibit 1
(1 of 2)

Activity:	Written by:	Date Prepared:
	1.	Submitted by:
	2.	Reviewed by:
	3.	Approved by:
	4.	

Personnel and Skill:

Tools and Equipment:

Materials and Supplies:

Safety Equipment:

Environmental Consideration: NEPA Compliance Date

Job Coordination: (Irrigation Districts, Utility Co., Other Branches, etc.)

Exhibit 1
(2 of 2)

Sequence of Basic Job Steps	Potential Accidents or Hazards	Recommended Safe Job Procedure

Exhibit 2
(1 of 2)

Rio Grande Project

DIVERSION DAMS

Equipment and Inspection Check List

ITEM OF INSPECTION	INSPECTION CARD CODE	INSPECTION INTERVAL	PROCEDURE REFERENCE
Slide gate (general inspection)	HDG*#a-12-? SLG*#a-12-?	Annual	1.01
Radial gate (general inspection)	SLG*#b-12-? RVG*#a-12-?	Annual	1.02
Lubrication of all movable mechanical parts	HDG*#b-2-? SLG*#c-2-? RVG*#b-2-?	Annual	1.03
Canal headgates, sluice gates and river gates (operation test)	HDG*#c-10-? SLG*#d-10-? RVG*#c-10-?	Annual	1.04
Gas powered gate operator (general inspection)	MOTG*a-12-?	Annual	1.05
Gas powered gate operator (air filter)	MOTG*b-2/4/6/8-?	2 month	1.06

Exhibit 2
(2 of 2)

PROCEDURE REFERENCE

1.01 Slide gate (general inspection)	Annual inspection.-- Visual inspection of seals, stems, gates, gate braces and gate guides for signs of cracks, corrosion, deterioration, etc. Close inspection of gate pedestals, neoprene pads and all movable mechanical parts for signs of corrosion, broken welds, loose bolts, cracks, deterioration, etc.
1.02 Radial gate (general inspection)	Annual inspection.--Visual insepction of seals, gates, gate braces and gate guides for signs of cracks, corrosion, deterioration, etc. Close inspection of hoist stands, sheaves, drums and all movable mechanical parts for signs of corrosion, broken welds, loose bolts, cracks, deterioration, etc. Close inspection of all wire ropes for abrasion, wear, fatigue, corrosion, improper reeving, kinking and binding. Check to see if all wire ropes have the correct number and placement of wire rope clips and thimbles.
1.03 Lubrication of all movable mechanical parts	Annual inspection.--Lubricate all gate stems, grease fitting, wire ropes and all movable mechanical parts with appropriate lubricants.
1.04 Canal headgates, sluice gates and river gates (operations test)	Annual inspection.-- Operate all canal and sluice gates and river gates, under normal operating conditions (average normal head), from the fully closed position to the fully open position and back to fully closed position.
1.05 Gas powered gate operator (general inspection)	Annual inspection.-- Inspect and clean crankcase breather, air filtering element and cooling fans. Change crankcase oil and replace oil in clutch and reduction gear housing. Replace oil filter and lubricate distributor shaft and cam. Regap or replace spark plugs, breaker points and adjust timing. Inspect starting motor for loose connections and clean bendix with kerosene. Check compression in motor.
1.06 Gas powered gate operator (air filter)	2 month inspection.-- Inspect and clean air filter and replace oil in filter bowl.

Exhibit 3

[illegible]
BUREAU OF RECLAMATION

FILE NO ______________________

____

EQUIPMENT ______________________

FACILITY ______________________ LOCATION ______________________

INSPECTIONS FREQUENCY ______________________

EQUIPMENT DATA

Specification/Solicitation ______________________ Date Furnished ______________________

Supplier ______________________ Type ______________________

MFR ______________________ Size ______________________

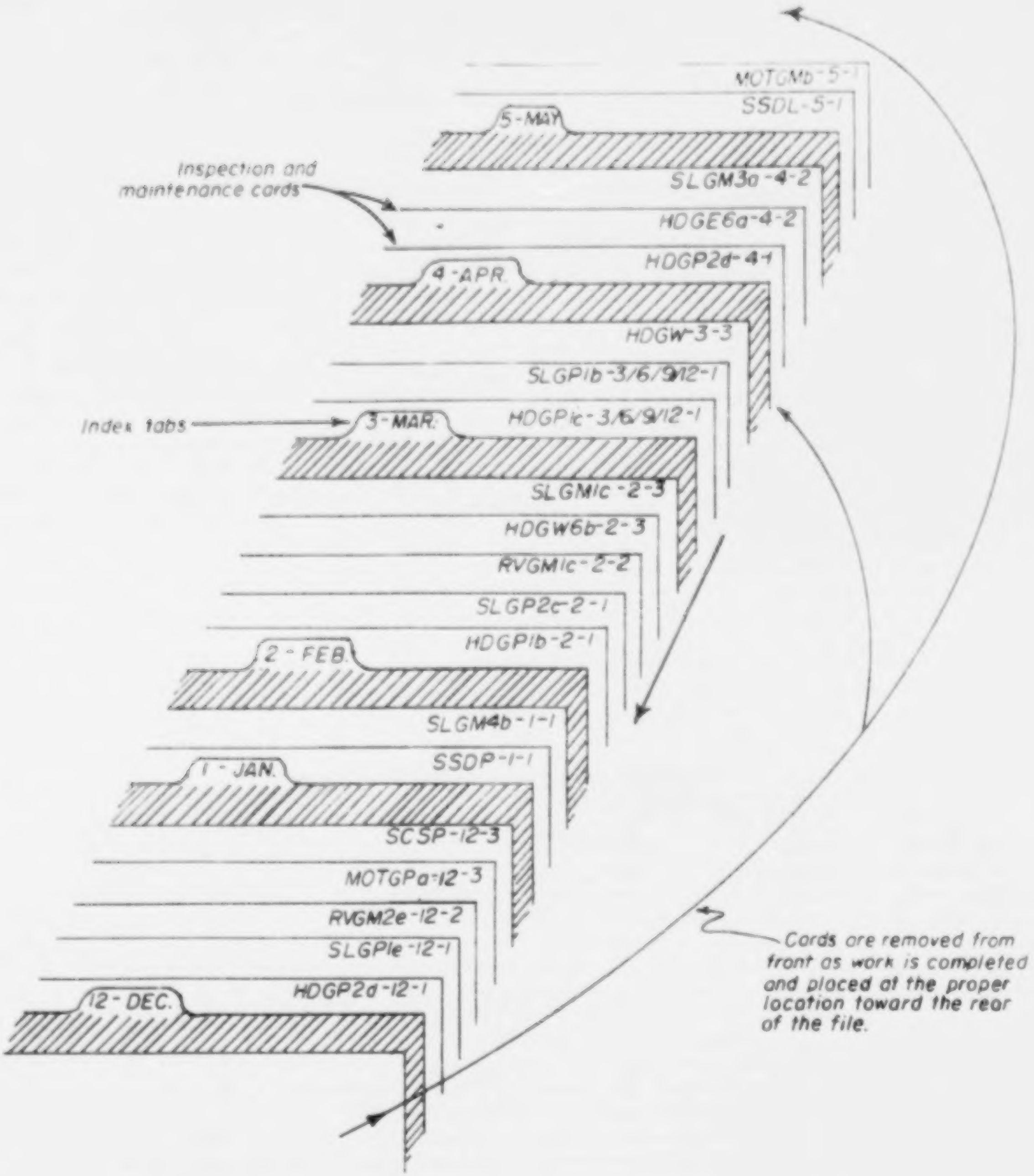
Model or Style No. ______________________

FRONT

INSPECTION AND MAINTENANCE RECORD

DATE	INSPECTED BY	CONDITION FOUND OR DESCRIPTION OF WORK DONE	HOURS	MATERIAL

Exhibit # 4


MOTGMb-5-1
SSDL-5-1
5-MAY
Inspection and maintenance cards
SLGM3a-4-2
HDGE6a-4-2
HDGP2d-4-1
4-APR.
HDGW-3-3
SLGPIb-3/6/9/12-1
HDGPIc-3/6/9/12-1
Index tabs
3-MAR.
SLGMIc-2-3
HDGW6b-2-3
RVGMIc-2-2
SLGP2c-2-1
HDGPIb-2-1
2-FEB.
SLGM4b-1-1
SSDP-1-1
1-JAN.
SCSP-12-3
MOTGPa-12-3
RVGM2e-12-2
SLGPIe-12-1
HDGP2d-12-1
12-DEC.

Cards are removed from front as work is completed and placed at the proper location toward the rear of the file.

FILE No.

1st. Symbol = Equipment designation symbol followed by a location designation, unit number and activity code.

2nd. Symbol = Month due

3rd Symbol = Week due.

ROTATING CARD FILE SYSTEM FOR MAINTENANCE SCHEDULES

Exhibit 5

EQUIPMENT DESIGNATION SYMBOL

Item	Code
air compressor	AIR
aqueduct	AQU
battery	BAT
bridge	BRG
building	BUI
canal	CAN
check gate	CKG
check structure	CKS
communication	COM
concrete structure	COS
crane	CRA
dam	DAM
desilting works	DEW
dike	DIK
diversion dam	DVD
drop structure	DRS
elevator	ELV
equipment light	ELT
equipment heavy	EHY
fish facility	FIF
flume	FLU
generator	GEN
grounds	GRD
head gate	HDG
instrumentation	INS
levees	LEV
motor	MOT
motor (gas)	MOTG
motor (electric)	MOTE
outlet works	OLW
outlet works gate	OWG
penstock	PEN
penstock gate	PEG
penstock valve	PEV
pump	PUM
pumping plant	PUP
radial gate	RDG
recreation area	RCA
reservoir	RES
river gate	RVG
roads	RDS
security and safety devices	SSD
sewage	SEW
siphon	SIP
sluice gate	SLG
spillway gate	SPG
spillway	SPY
vehicle	VEH
ventilation	VEN
wasteway	WAS
wells	WEL

7-2053 (5-81)
Bureau of Reclamation

WORK ORDER

EXHIBIT 6

W. O. IDENT.

	NAME	DATE
APPROVED		
AUTHORIZED		

REQUESTED BY:

DATE: WORK REQUEST IDENT.:

PLANNER: DATE:

ROUTE TO	REQ'D	INITIALS	DATE

JOB TITLE:

PRIORITY CODE ☐

10. EMERGENCY (NOW)
11. START NEXT WORK DAY
20. URGENT (START WITHIN ONE WEEK)
30. NORMAL (________) START DATE
40. NEXT SHUT-DOWN, OUTAGE, OR OVERHAUL

ACTIVITY CODE	CENTER	FAC	AREA	PRIM	IDENTIFICATION	COMP

WORK NAME AND LOCATION

JOB COST NUMBER

DESCRIBE WORK REQUESTED AND SPECIAL REQUIREMENTS:

																TOTAL
TIME SLOT																
PERSONNEL CODE																▨
EST. NO. PEOPLE																
PERSON-HOURS																
ACT. PERSON HRS.																

HAZARD ANALYSIS:
BY: DATE:

DATE JOB COMPLETED:

MAINTENANCE ACTION TAKEN, RECOMMENDATIONS, PROBLEMS ENCOUNTERED, AND APPROPRIATE SIGNATURE(S):

REASON FOR DOING WORK

	PRIMARY	SECONDARY
CAUSE CODE		

TOTAL ESTIMATE COST:
$

LABOR COST:
$

SUPPORT COST:
$

COST OF MATERIALS:
$

TOTAL JOB COST:
$

Appendix

Sample MMS

Percha, Leasburg, and Mesilla Diversion Dams

Rio Grande Project

A manual MMS has been developed for use on the Rio Grande Project to perform preventive maintenance on three diversion facilities.

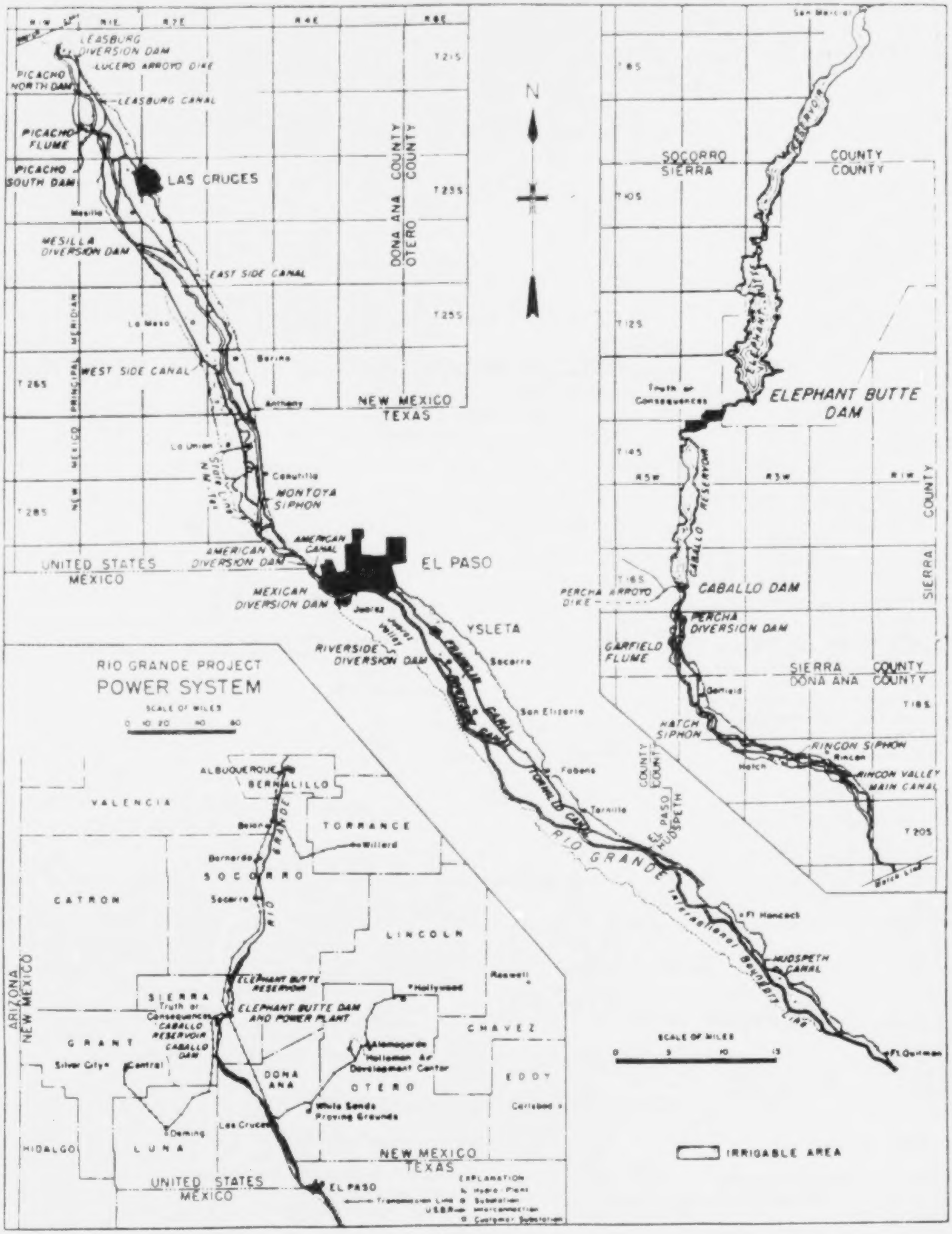
The most northerly diversion facility of the project is the Percha Diversion Dam which is located 2 miles south of Caballo Dam on the Rio Grande (see location map). The facility was constructed in 1918 and is a concrete ogee weir structure with two radial sluice gates on the Rio Grande and eight cast iron slide gates for water deliver to the Arrey Canal. All gates at this facility, except for two slide gates, are operated with a 4-cycle, 2-cylinder gasoline-powered engine. The two slide gates that are not operated by this engine are electrically operated.

The next diversion facility of the project is the Leasburg Diversion Dam. This diversion dam is located on the Rio Grande about 15 miles northwest of Las Cruces, New Mexico (see location map). This facility was constructed in 1907 and the crest was raised 1.25 feet in 1919. The structure is a concrete ogee weir structure with three wooden slide gates on the Rio Grande and seven steel slide gates for water delivery to the Leasburg Canal. All gates are powered by a 4-cycle, 2-cylinder gasoline-powered engine.

The third diversion facility of the project is the Mesilla Diversion Dam which is located 6 miles south of Las Cruces, New Mexico, on the Rio Grande. This facility was constructed in 1916 and the crest was raised 1.66 feet in 1940. The structure is a concrete weir-radial gate structure with 13 radial gates on the Rio Grande, and 2 canal headings with 8 cast iron slide gates on the west side, and 6 cast iron slide gates on the east side.

All three diversion dams are maintained by a maintenance crew of four, including one supervisor. One day of the week the field supervisor reports to the main office to discuss his crews' activities in the field and receive any work orders (attached) or copies of Inspection and Maintenance Record Cards (attached) for the following month. Upon completion of the work, the appropriate work order form or copy of the Inspection and Maintenance Record Card is filled out and returned to the main office. The work orders are filed by facility name, while the information on the copies of the Inspection and Maintenance Record Cards is transferred to the original card, a report is made, the card is filed by month, and a copy of the card is filed by facility name.

A work order can be turned in by anyone involved in project O&M. The crew foreman is responsible for preparing a job plan and safety analysis for each job and recording labor and materials used on the reverse side of the work order form. All original documents are kept in the El Paso Office and are monitored by the Engineering and Maintenance Branch. Several sample work order forms and Inspection and Maintenance Record Cards are attached.


LEASBURG DIVERSION DAM
LUCERO ARROYO DIKE
PICACHO NORTH DAM
LEASBURG CANAL
PICACHO FLUME
PICACHO SOUTH DAM
LAS CRUCES
Mesilla
MESILLA DIVERSION DAM
EAST SIDE CANAL
La Mesa
Berino
WEST SIDE CANAL
Anthony
NEW MEXICO PRINCIPAL MERIDIAN
La Union
Canutillo
MONTOYA SIPHON
AMERICAN CANAL
AMERICAN DIVERSION DAM
UNITED STATES
MEXICO
EL PASO
MEXICAN DIVERSION DAM
Juarez
YSLETA
RIVERSIDE DIVERSION DAM
Socorro
San Elizario
Fabens
Tornillo
RIO GRANDE
Ft Hancock
HUDSPETH CANAL
Ft Quitman
DONA ANA COUNTY
OTERO COUNTY
NEW MEXICO
TEXAS
T 21 S
T 23 S
T 25 S
T 26 S
T 28 S
R 1 W
R 1 E
R 2 E
R 4 E
R 6 E
N
San Marcial
SOCORRO COUNTY
SIERRA COUNTY
ELEPHANT BUTTE RESERVOIR
Truth or Consequences
ELEPHANT BUTTE DAM
CABALLO RESERVOIR
PERCHA ARROYO DIKE
CABALLO DAM
PERCHA DIVERSION DAM
GARFIELD FLUME
Garfield
SIERRA COUNTY
DONA ANA COUNTY
HATCH SIPHON
Hatch
RINCON SIPHON
Rincon
RINCON VALLEY MAIN CANAL
EL PASO COUNTY
HUDSPETH COUNTY
T 8 S
T 10 S
T 12 S
T 14 S
T 16 S
T 18 S
T 20 S
R 5 W
R 3 W
R 1 W
SCALE OF MILES
0 5 10 15
IRRIGABLE AREA
RIO GRANDE PROJECT
POWER SYSTEM
SCALE OF MILES
0 10 20 40 60
ALBUQUERQUE
BERNALILLO
VALENCIA
Belen
TORRANCE
Willard
Bernardo
SOCORRO
Socorro
CATRON
LINCOLN
ARIZONA
NEW MEXICO
ELEPHANT BUTTE RESERVOIR
SIERRA
Truth or Consequences
CABALLO RESERVOIR
CABALLO DAM
ELEPHANT BUTTE DAM AND POWER PLANT
Hollywood
Roswell
CHAVEZ
GRANT
Silver City
Central
Alamogordo
Holloman Air Development Center
DONA ANA
OTERO
EDDY
Carlsbad
White Sands Proving Grounds
Deming
Las Cruces
HIDALGO
LUNA
NEW MEXICO
TEXAS
UNITED STATES
MEXICO
EL PASO
EXPLANATION
Hydro Plant
Transmission Line
Substation
USBR Interconnection
Customer Substation

Rio Grande Project

Outlet works: Uncontrolled concrete conduit through base of dike.

Capacity at El. 3930	180 ft³/s

Diversion Facilities

PERCHA ARROYO DIKE[6]

Type: Rock-faced earthfill
Location: On Percha Arroyo, 1 mi west of Caballo Dam.
Year completed: 1939
Dimensions:

Structural height	29 ft
Hydraulic height	10 ft
Total crest length	2,489 ft
Crest elevation	4200.0 ft
Volume	193,000 yd³

Spillway: None
Headworks: Flood diversion channel, no gates, highway bridge and drop chute into Caballo Reservoir.

Diversion capacity	30,000 ft³/s

---

[6] Dike constructed to divert storm runoff into Caballo Reservoir; no direct connection with irrigation system.

PERCHA DIVERSION DAM

Type: Concrete ogee weir, embankment wings
Location: On the Rio Grande, about 2 mi south of Caballo Dam.
Year completed: 1918
Dimensions:

Structural height	18.5 ft
Hydraulic height	8 ft
Weir crest length	350 ft
Total crest length	2,720 ft
Crest elevation	4103.0 ft
Volume	43,200 yd³

Spillway: Overflow weir, 2 radial sluice gates, each 20- by 8-ft.
Headworks: Rincon Valley Main Canal headworks at west abutment; 8 slide gates, each 4.3 by 3.75 ft.

Diversion capacity	350 ft³/s

LEASBURG DIVERSION DAM

Type: Concrete ogee weir, embankment wings.
Location: On the Rio Grande, about 15 mi northwest of Las Cruces, N. Mex.
Year completed: 1907. Crest raised 1.25 ft in 1919.
Dimensions:

Structural height	10 ft
Hydraulic height	7 ft
Total crest length, dam, including weir	2,865 ft
Weir crest length	600 ft
Weir crest elevation	3922.25 ft
Volume	22,500 yd³

Spillway: Overflow weir, 3 slide sluice gates, each 5 by 8 ft.

Capacity[7]	17,000 ft³/s

Headworks: Leasburg Canal headworks at abutment; 7 slide gates 5 by 6.75 ft.

Diversion capacity	625 ft³/s

MESILLA DIVERSION DAM

Type: Concrete weir, radial gate structure
Location: On the Rio Grande, 6 mi south of Las Cruces, N. Mex.
Year completed: 1916. Crest raised 1.66 ft in 1940.
Dimensions:

Structural height	22 ft
Hydraulic height	10 ft
Weir crest length	303 ft
Crest elevation	3819.83 ft
Volume	2,900 yd³

Spillways: Nine radial gates, each 21.58 by 6 ft; 4 radial gates, each 21.58 by 8.42 ft.

Capacity[7]	15,000 ft³/s

Headworks: Canal headworks at each abutment. 8 slide gates, each 4.33 by 3.75 ft at west end; 6 slide gates, each 4.33 by 3.75 ft at east end.
Diversion capacity

West side	650 ft³/s
East side	300 ft³/s

AMERICAN DIVERSION DAM[8]

Type: Radial gate structure between river levees
Location: On the Rio Grande at El Paso, Tex.
Year completed: 1938
Dimensions:

Structural height	18 ft
Hydraulic height	5 ft
Weir crest length	286 ft
Crest elevation	3683.5 ft
Volume	2,900 yd³

Spillway: Thirteen radial gates, each 20 by 7.5 ft

Capacity[7]	12,000 ft³/s

Headworks: American Canal headworks at east abutment; 2 radial gates, each 20 by 11 ft.

Diversion capacity	1,200 ft³/s

RIVERSIDE DIVERSION DAM

Type: Concrete weir, radial gate structure
Location: On the Rio Grande, 15 mi southeast of El Paso.
Year completed: 1928
Dimensions:

Structural height	17.5 ft
Hydraulic height	8 ft
Weir crest length	267 ft
Crest elevation	3621.07 ft
Volume	2,500 yd³

Spillway: Six radial gates, each 16 by 8.17 ft, overflow weir.

Capacity[7]	11,000 ft³/s

Headworks: Riverside Canal headworks at east abutment; 5 radial gates, each 16 by 6 ft.

Diversion capacity	900 ft³/s

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[7] These are the flood discharges which the International Boundary and Water Commission is using for the Rio Grande Channelization Project at the indicated points and are not necessarily the maximum which the dams will pass safely, nor which they were designed to pass.

[8] American Diversion Dam and Canal constructed and operated by American Section, International Boundary and Water Commission, United States-Mexico, for the diversion and allocation of water in accordance with treaty between the United States and Mexico.

Carriage Facilities

RINCON VALLEY MAIN CANAL

Location: From Percha Diversion Dam generally southeast along Rio Grande.
Construction period: 1916-19

Length	28.1 mi
Diversion capacity	350 ft³/

Work Action No 157

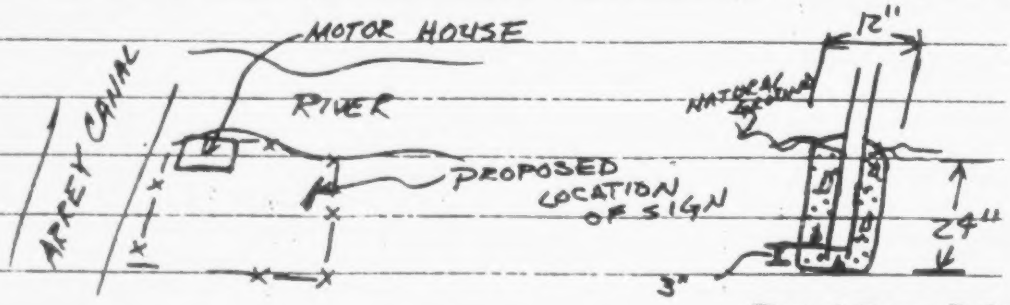
Work Order No. 146

Priority Code 3
(1) Emergency
(2) Urgent (within one week)
(3) Normal-Date required SEPT 27, 1985
(4) Next Shutdown (winter Maintenance)
(5) Inclement Weather Activities

Work Name and Location PERCHA DIVERSION DAM

Describe Work Requested, Recommended, and Justification

1. INSTALL SIREN SIGN ~~on~~ IN FENCED AREA



BORY POST 2ft IN CONCRETE.

Work Authorized by
Date 9/9/85

Maintenance Action Taken, Recommendation, Problems Encountered (include number of people, hours worked, craft used, etc.)

Pick up pipe at Bergman Steel - Transport to job Site - Dig anchor hole and fill with Concrete - look for good gravel and haul to job Site to Mix Concrete - Set steel Posts in Concrete and leave to set over night - attatche sign to Posts by means of welding. 2 Men two Trips to Percha work 16 hrs ea to include travel time Dully - welder - Cutting Torch

Completed by

Date Completed 9-12-85

Work Action No. 106

Work Order No. 106

Priority Code 3
(1) Emergency
(2) Urgent (within one week) 6-30-85
(3) Normal-Date required ~~Mar 29, 1985~~
(4) Next Shutdown (winter Maintenance)
(5) Inclement Weather Activities

Work Name and Location MESILLA AUTOMATION

Describe Work Requested, Recommended, and Justification

① INSTALL GATE CONTROLLER
2 Control Pannell
3. Adjust wire ropes to operate gate

Work Authorized by
Date 3-11-85

Maintenance Action Taken, Recommendation, Problems Encountered (include number of people, hours worked, craft used, etc.)

① Drill holes on base of Controller unit, hoist in place with Nuckle boom Crane and secure to anchore bolts - Connect gate wire ropes to Power drums. 4-2-85
② Drill holes in Concrete for ancher bolts and place Control Pannel in Position, Secure. 4-3-85
③ Replace broken bolt in Power drum and Cable tie [illegible] adjust wire Ropes to full [illegible] and make gate operational. [illegible] 5-8-85
Nuckle Boom-Crane, Hilti drill-Power drill-Cutting Torch-Hand tools-[illegible]

Completed by Date Completed 6-17-85

Work Action No. 153

Work Order No. 149 & 150

Priority Code 3
(1) Emergency
(2) Urgent (within one week)
(3) Normal-Date required Oct. 31, 1985
(4) Next Shutdown (winter Maintenance)
(5) Inclement Weather Activities

Work Name and Location MESILLA DIVERSION DAM

Describe Work Requested, Recommended, and Justification

1. INSTALL FIRST AID KIT & FIRE EXTINGUISHER ON WALL IN MESILLA BLOCK HOUSE

2. PAINT METER HOUSING YELLOW & BLACK STRIPES @45°

Work Authorized by
Date 9/24/85

Maintenance Action Taken, Recommendation, Problems Encountered (include number of people, hours worked, craft used, etc.)

trip to Mesilla Dam paint meter box yellow and let dry - install first aid Kit and fire extinguisher bracket to hang fire extinguisher.
two men worked 3 hours each to include travel time - used hand tools, paint and provided screws.

Completed by Carlos Madrid

Date Completed 10-3-85

[illegible]
BUREAU OF RECLAMATION

FILE NO. HDGE6a- 12 -3

EQUIPMENT HEADGATE ON EASTSIDE CANAL

FACILITY Mesilla Diversion Dam LOCATION 6th gate of north dam INSPECTIONS FREQUENCY 12 months

EQUIPMENT DATA

Specification/Solicitation

Date Furnished

Supplier

Type

MFR

Size

Model or Style No.

INSPECTION AND MAINTENANCE RECORD

DATE	INSPECTED BY	CONDITION FOUND OR DESCRIPTION OF WORK DONE	HOURS	MATERIAL
12-19-84		PROCEDURE 1.01 SEAL DETERIORATED Angle iron missing - Pedestal cracked Need Paint - Neoprene pad missing	1/4	

7-VC-140 (8-84)
BUREAU OF RECLAMATION

FILE NO. [illegible]-12

EQUIPMENT RIVER GATES

FACILITY MESILLA DIVERSION DAM LOCATION 4th GATE FROM WEST END INSPECTIONS FREQUENCY 12 MONTHS

EQUIPMENT DATA

Specification/Solicitation ____ Date Furnished ____

Supplier ____ Type ____

MFR ____ Size ____

Model or Style No. ____

INSPECTION AND MAINTENANCE RECORD

DATE	INSPECTED BY	CONDITION FOUND OR DESCRIPTION OF WORK DONE	HOURS	MATERIAL
12-18-84	M Sierra	PROCEDURE 1.01 SLIGHT CORROSION ON GATE Thimbles are missing. Wire rope needs lubrication.	1/4	

PVC-14C (8-84)
BUREAU OF RECLAMATION

FILE NO. [illegible]-6/12-2

EQUIPMENT GAS POWER GATE OPERATOR

FACILITY LEASBURG DIVERSION DAM LOCATION West Bank of River at Dam Site

INSPECTIONS FREQUENCY 6 months

EQUIPMENT DATA

Specification/Solicitation 336855

Date Furnished

Supplier serial no. 5088934

Type wisconsin air cooled

MFR Teledine

Size 4 cylinder - 8 cycle

Model or Style No. VH4D

INSPECTION AND MAINTENANCE RECORD

DATE	INSPECTED BY	CONDITION FOUND OR DESCRIPTION OF WORK DONE	HOURS	MATERIAL
12-4-84	M. Silva	Procedure 1.0[illegible]	1	None
6-1-85	I. Ortega	No adjustment required	1/2	—

Review of Facilities

The Bureau of Reclamation has designed and constructed about 350 storage dams, 250 diversion dams, and 250 separate water distribution projects and other appurtenant facilities. About one-third of the storage dams (usually multipurpose dams) are operated by the Bureau and three-fourths by operating entities. Most diversion dams and all distribution facilities are transferred to an operating entity for operation and maintenance. Some major canals which are multipurpose are also "reserved" for operation by the Bureau. In either case, a periodic formal examination referred to as the RO&M (Review of Operation and Maintenance) Program is performed on these facilities. This program was initiated because it was observed that a general deterioration of facilities was occurring due to inadequate operation and maintenance programs.

The RO&M examinations are separated into two categories - major and minor examinations. The major examinations include all storage dams, some diversion dams, and major pumping plants. The minors include all distribution facilities (canals, laterals, structures, smaller pumping plants, etc.) and most of the diversion dams. The major examinations are performed by a team from the Denver Office every 6 years and by regional teams on a 3-year basis. This process results in a formal examination every 3 years. The minor examinations are done every 3 years by regional personnel.

A report is written on each examination with the general conditions of the structures and deficiencies noted and recommendations made for rectification. In addition, less formal examinations are performed annually or more frequently by Bureau and water user field personnel to evaluate operation and maintenance on an ongoing basis.

The requirements for these examinations are established by Bureau policy and their main purpose is to assure that the facility is operated and maintained properly to accomplish its intended purpose and project payback. In addition, other benefits such as training of young engineers; feedback on design, construction, operation and maintenance procedures; and compilation of data are realized by the Bureau and the operator.

These formal examinations include examining and operating (if possible) all facilities and evaluating the overall operation and maintenance program being carried out. A general checklist is provided as figure 1 which is used by examiners to assure completeness.

After the examination is finished, a report is written to document the findings. The general condition of the facilities, appropriateness of operational and maintenance procedures, deficiencies, and recommendations for correcting the problems are documented in the report. The format of the reports should include, as a minimum, the following items:

Major Examinations

- Reference data (drawings and other pertinent information)
- Structural performance data
- Previous examinations including underwater/dewatered examinations
- Date of examination and names of examining party
- Facilities examined
  - Dam
  - Spillway(s)
  - Outlet works
  - Other
- Operations
  - Standing operating procedures
  - Communications
  - Logbook
  - Access
  - Landslide
  - Dam tender's training
- Status of previous recommendations
- New recommendations
- Signatures
  - Examiners
  - Supervisors
- Photographs to illustrate status and findings

Minor Examinations

- General Description
  - Location of facilities
  - Acres served
  - Type of facilities
  - Statistics
- Special Information
  - O&M assessment
  - Reserve fund
  - Equipment replacement fund
  - Number of employees
  - Number of directors
- Previous Examination Date(s)
- Date of Examination
- Examining Party
- Operational Status at Time of Examination
- Facility Headings
  - Diversion dam
  - Headworks
  - Canal
  - Turnouts
  - Check structures
  - Overchutes
  - Siphons
  - Tunnels

Facility Headings - continued

- Bridges

Pumping plants

Closed conduit systems

- Pipe
- Vaults
- Pumping plants
- Reservoirs
- Surge tanks

Weed control

District headquarters and equipment

Conclusions

Explanation of Recommendation Categories

- Status of Previous Recommendations
- New Recommendations

Responsibility for completion of the recommendations is in the hands of the Bureau official (regional director) who is in charge of operation and maintenance of the facility. Repayment contracts between the Bureau and the operating entity and Bureau policy provide for completion of the recommendations. If the work is not done willingly and in a timely manner, the Bureau can either do the work or have it done and bill the operating entity or, as a last resort, take over operation of the facility. These two measures are infrequently implemented. The usual procedure is to discuss the potential problems with operating personnel and provide reasons why recommendations are being made and why the work should be done.

Recommendations are made based primarily on the judgment of the examination team members. Judgment of the examiner usually takes precedence over standardized procedures if questions arise. Controversial issues are discussed with higher management; however, emphasis is placed on providing flexibility and independence to the examiners.

Recommendations for correcting deficiencies are made in one of the following three categories:

Category 1

Recommendations involving the correction of severe deficiencies where immediate and responsive action is required to ensure structural safety and operational integrity of a facility.

Category 2

Recommendations covering a wide range of important matters where action is needed to prevent or reduce further damage or preclude possible operational failure of a facility.

Category 3

Recommendations covering less important matters, but believed to be sound and beneficial suggestions to improve or enhance the operation and maintenance of a project or facility.

All new recommendations are listed in the report and identified as to year made, category, and individual identity (i.e., 84-2-A). Previous recommendations not completed are also reported on and listed under that heading. A computer printout is utilized to document the status of recommendations. A schedule for completion of recommendations is requested from the operating entity at the time of report transmittal to them.

The examination team consists of a team leader and other personnel who have the combined expertise of operation and maintenance, concrete or embankment dam design, and mechanical design (two to three persons). Team assignments are made on a case-by-case basis depending on site-specific conditions. Team members receive on-the-job training by participating in examinations prior to becoming one of the primary team members. Other experience should be gained in their primary field of endeavor before becoming a team member. Workshops are held periodically for examiners to discuss problem areas and to provide continuity and uniformity on a Bureau-wide basis. In addition to the team members, other personnel from Bureau field offices, and sometimes other Federal and State agencies, participate in these examinations. Their role is usually as observers to gain knowledge of dam deficiencies and overall conditions. Occasionally, Safety of Dams personnel also participate in these examinations.

In addition to the examination needs, there are other requirements associated with the safe operation and proper maintenance of storage dams. Each dam is required to have an SOP (Standing Operating Procedures) document. This document is kept at the dam and provides the operator with detailed instructions on the safe and proper operation of the facility. In general, it includes instructions on emergency preparedness, structural, mechanical, electrical, reservoir operations (hydrological), inundation maps, and other pertinent information. A "Guide for Preparation of SOP's for Bureau of Reclamation Dams and Reservoirs" is used in preparing the SOP for each dam. General standards for operation and maintenance of Bureau dams are outlined in this "Guide." The SOP's are partially reviewed on a random basis during the RO&M examination for adequacy, and dam operators are required to perform procedures as specified by the examination team.

Training is required for all operators of Bureau dams prior to independent operation with a refresher course at least every 3 years. Requirements and content of this training are outlined in "Training for Dam Operators - A Manual for Instructors," United States Department of the Interior, Bureau of Reclamation. The training consists of 1 to 2 days of classroom-type activity on general aspects of dam design, operation, maintenance, emergency preparedness, hydrology, instrumentation, and other related factors, and onsite training for each operator of dams. The SOP is used as a guide for the onsite training.

As a result of the RO&M Program examinations, numerous problems and deficiencies have been identified at Bureau-constructed projects. Identification and rectification of these problems have resulted in exchanges of ideas between operations personnel on problem solutions and operation and maintenance procedures and in a higher standard of operation and maintenance.

Some of the general and unique problems which have been identified under the RO&M Program and other onsite visits will be discussed during this session. Participants are encouraged to come prepared to discuss operation and maintenance problems and solutions which they have experienced or with which they are familiar.

Figure 1

CHECKLIST FOR EXAMINATIONS OF MAJOR STRUCTURES

Concrete Dam

- Upstream face
- Downstream face
- Crest
  - Roadway
  - Walks
  - Parapet wall
  - Lighting, etc.
- Galleries
  - Concrete
  - Metalwork
  - Electrical
  - Ventilation
  - Drains and drainage
- Elevator shaft
  - Metalwork
  - Equipment
  - Safety inspection
- Abutments
- Foundation at downstream toe of dam
- Leakage around dam
  - Location
  - Amount

Leakage around dam - continued

- Measurement methods

Performance instruments and devices

- Uplift measurements
- Drain flow

Earth Dam

- Upstream face
  - Riprap
  - Erosion - Beaching
  - Vegetative growth
  - Settlement
  - Debris
- Downstream face
  - Rock
  - Vegetative growth
- Crest
  - Roadway
  - Guardrails
  - Curb
  - Parapet wall
  - Settlement
  - Lighting
- Abutments
- Seepage and drainage
  - Location
  - Toe drain

Seepage and drainage - continued

- Measurement
  - Method
  - Amount
  - Change in flow
  - Records

Performance instruments

- Surface settlement points
- Piezometer well
- Readings
- Other

Spillway

- Approach channel
  - Channel
  - Log boom
- Control structures
  - Crest
  - Walls
  - Apron
- Chute
  - Walls
  - Floor
  - Drains
- Stilling basin
  - Walls
  - Floor
- Outlet channel
  - Riprap
  - Erosion
  - Vegetation
- Structural
  - Hoist deck
  - Bridge

Spillway - continued

- Gates
- Mechanical features
  - Hoists
  - Cables
  - Gates
  - Protective coatings
- Electrical features
  - Power supply
  - Standby power
- Posted operating instructions
- Exercising frequency
- Automatic controls
- Ice prevention system

Other

Outlet Works

- Inlet structure
  - Trashracks
  - Concrete
  - Gate chamber
    - Gates
    - Operation at time of examination
    - Exercising frequency
    - Mechanical
    - Electrical
    - Protective coatings
    - Posted operating instructions
  - Ventilation
  - Seepage
  - Concrete
  - Access tunnel
    - Concrete
    - Metalwork
  - Outlet conduit
    - Metalwork
    - Protective coatings
    - Concrete
    - Cavitation

Outlet works - continued

- Control facilities
  - Control house
    - Structural condition
      - Roof
      - Walls
    - Housekeeping
    - Metalwork
    - Protective coatings
    - Gates
      - Operation at time of examination
      - Exercising frequency
      - Mechanical
      - Electrical
      - Protective coatings
      - Posted operating instructions
- Chute
  - Floor
  - Walls
  - Drains
- Stilling basin
- Outlet channel
  - Vegetation
  - Gravel bars, etc.

Operations

Attendance at dam

Dam tender's training

Residence

Standing Operating Procedures and Designers' Operating Criteria

	SOP	DOC
Issue and revision dated		
Is copy at dam current?		
Does SOP meet current guidelines?		
Are SOP supporting documents available?		
Are instructions adequate?		
Are instructions understood?		
Any changes needed?		

Operating Log

Maintained at dam?

Certification of SOP review and dam tender's training (dated and signed by dam tender and supervisor)

All RO&M team members sign and date

Team leader verify (sign and date)

1. Current SOP is on hand and revisions inserted
2. Operating log signed and dated by dam tender and supervisor verifying SOP certification requirements
3. All operating procedures observed during review were in accordance with SOP

4. All SOP supporting documents available

5. Recommendation to correct deficiency in any of the above items will be made

Communications

  Type

    Normal

    Standby

  Adequate

Auxiliary power

  Test during examination

  Condition

  Adequate

Access roads

  Adequate under adverse conditions

Oil containment

  SPCC Plan

Landslides

  Restricted

  Signs

Major Pumping Facilities

- Structural features
  - Building (wood, concrete, metal construction)
    - Walls
    - Roof
    - Decks
    - Drainage
    - Crane
    - Doors, windows
    - Ventilation
    - Lighting
    - Safety
- Forebay conditions
  - Inlet channel
  - Concrete
  - Riprap
  - Debris or silt bars
  - Trashracks
  - Stoplogs and grooves
  - Sump conditions
  - Other metalwork
- Pumping units
  - Maintenance procedures
  - Vibration
  - Cavitation
  - Lubrication

Pumping units - continued

- Bearing temperatures
- Packing box leakage
- Paint deterioration
- Unit mounting and foundation
- Low water cutoff system
- Pump priming system
- Other operating difficulties

Discharge valves (manual, motor, or hydraulic-operated)

- Operation
- Vibration
- Maintenance procedures
- Lubrication
- Paint deterioration
- Unit mounting and foundation

Electrical control equipment

- Housekeeping (clean and dry)
- Protective coatings
- Maintenance procedures
- Other operating difficulties

Afterbay conditions

- Discharge piping
  - Interior
  - Exterior
- Anchor blocks

Afterbay conditions - continued

- Discharge boxes
- Flap valves
- Other metalwork

Powerhouse or Pumping Facilities

- Structural features
  - Building
    - Roof
    - Decks
    - Parapets
    - Walls
  - Substructure
  - Superstructure
- Forebay conditions
- Afterbay conditions

Bridge

Name ______________________________ Group __________

Type I. Intermediate Inspection

This inspection should not be considered as an inspection in depth. The purpose of the Type I inspection is primarily to insure the safety of the bridge user. This entire inspection should be properly documented. This inspection can be limited to:

- Bridge supports
  - Foundations
  - Substructures - Piers
  - Bridge bearings
  - Moving parts
  - Accumulation of bird nests, etc.
  - Visual inspection of scour protection
  - Protective coatings
- Main supporting members
  - Deteriorated and/or damaged members
  - Live load capacity
- Bridge deck
  - Potholes
  - Deterioration
  - Unusual roughness
  - Drainage
  - Expansion joints
  - Guardrails
  - Signing
- General condition
- Vegetation at abutments and piers

CHECKLIST FOR EXAMINATIONS OF MINOR STRUCTURES

Diversion Dam

- Concrete
- Ogee weir
- Riprap
- Abutment walls
- Wingwalls
- Embankments
- Foundation at downstream toe of dam
- Fish ladders
- Leakage
- Embankments
  - Upstream
    - Vegetation
    - Riprap
  - Downstream
    - Vegetation
    - Rock
  - Crest
- Canal headworks
  - Concrete

- Canal headworks - continued
    - Gates
        - Coatings
        - Hoists
        - Operation
        - Leakage
    - Fish facilities
    - Guardrails
    - Outlet channel
    - Control structures
- Sluiceway
    - Concrete
    - Radial gate
        - Coatings
        - Hoists and cables
        - Operation
        - Leakage
    - Handrails
    - Approach channel
    - Outlet channel
- Gates
    - Condition
    - Hoists
    - Cables
- Standby power

Canal
- Subsidence
- Seepage
- Linings
  - Constructed joints
  - Random cracking
  - Joint sealers
  - Silting

Inlet Structure and Checks
- Structural features
- Radial gates
  - Hoists
  - Cables
  - Electrical features
  - Protective coatings
  - Controls

Turnouts
- Gates
- Structure

Bridges
- Deck
- Railings
- Structural features

Operating Roads

Vegetation

Safety Features

Pipeline System

- Valve stations
- Rights-of-way
- Seepage or wetness along alignment
- Construction activities in area
- Drainage
- Weed and tree control
- Pipeline cover material
- Siphons
- Surge tanks
- Regulating reservoirs
- Cathodic protection
- Capacity of pipeline
- Internal examination

Water Systems Operation and Maintenance Workshop - 1990

MAINTENANCE MANAGEMENT, PART 2
A Manager's Viewpoint of Project Maintenance

Session 1-2

Robert M. Fagerberg

Maintenance of irrigation distribution systems has taken a major shift in recognizing the need for effective utilization of water diverted for irrigation purposes. Today, the emphasis is on conservation and maintenance of the infrastructure from a national viewpoint. Project water users must be prepared to assume responsibility for ongoing maintenance and repair. Delayed maintenance (R&B) is not likely to be considered the responsibility of the Federal Government. Continued subsidies on water projects have created negative reactions on part of some of the congressional delegation, and environmental and wildlife groups to future support for water projects. Whether or not these reactions are justified, the handwriting is very clear and that is water users must take a more active and determined role to reduce reliance on federal subsidies. We have to be more self reliant in maintaining the project works.

The doctrines for allocation of water are being challenged by population growth, environmental issues, as well as in-stream flows, recreation, and wildlife. Therefore, how a project is managed determines the viability and survival of a project.

II. MAINTENANCE OF PROJECTS - REQUIREMENTS

A. Management Role

1. Every project must face certain limitations depending on geographic location, climatic limitation, marketing ability, size of farms, quality of soils, etc. Some of these limitations involve average value of crop revenue, ability to maintain systems with realistic assessments based on ability to pay and cropping intensities. Now with a renewed emphasis on conservation, projects must understand the importance of maintaining water rights and not ignore the history of water rights that establishes one's right to claim project water in times of shortages.

2. To develop strong operation, maintenance, management programs -- The main objective is to provide optimum system conditions for water users to realize maximum crop production on irrigated land. To achieve the optimum system conditions requires an organization with qualified management people with authority to carry out programs for maintenance of the system. This involves the need for 5 main goals:

(a) An informed water user organization
(b) Qualified employees
(c) Budget based on needed programs rather than past costs
(d) Funds to carry out an approved budget based on needs
(e) A Board of Commissioners who us willing to dedicate time to study project problems along with management

3. To visualize (Management)

(a) Problems today
(b) Problems tomorrow
(c) Develop a detailed plan of action to meet these problems, develop program costs - determine ability of water users to pay

4. To be accessible and visible to water users. Work closely with irrigation district boards and water users to provide maintenance programs which prevent need for premature rehabilitation and betterment programs, inefficient operation of the system, unnecessary system failure and encroachment on safety standards. I believe irrigation districts must assume a more active role in accepting responsibility for physical improvements to maintain their integrity and respect of those who support our needs on a federal basis. Good communications is absolutely essential between water users, employees, management, and boards. Communication eliminates many problems and misunderstandings and develops confidence among water users.

5. Courage to make decision (Management)

There are times when a decision has to be made which is unpopular. Before any of these decisions are made, management must have conducted a thorough analysis before recommending any action to the board. There must be a willingness to change, if change is indicated, because today courts are easily accessible on any decision that is not impartial, fair, or reasonable. Management must constantly remind themselves that they are part of the team along with the water users to efficiently operate the project. Some of the areas which involve tough decisions are:

(a) programs involving increasing O&M costs
(b) water rationing in time of shortages
(c) removing water for repair of system
(d) carrying out policies of the district
(e) establishing a program of conservation and distribution and on-farm efficiencies to improve overall district efficiency

In the final analysis, decisions are the responsibility of management and board, recognizing that any issue of controversial nature must be fully aired with water users before such decisions are made. Decisions made arbitrarily will eventually create loss of confidence by the water users.

B. Factors Affecting Maintenance Decisions

1. Cropping Intensities - Climate Conditions

Crop intensity determines the amount of time that the system is available for maintenance. Higher cropping intensity generally indicates that a more intense maintenance program of short time duration needs to be planned. In areas with a short frost-freeze season (100-140 days), cropping intensity is generally less than 90%. In the same context, this type of area, because of winter months, has a limited number of days for maintenance which increases cost because of costly preparation and specialized equipment needs. Certainly gross crop revenues also affect the amount of levy that can be made for O&M purposes. Generally, as a guideline, full assessment for both O&M and capital construction costs, should not exceed 10% of the average gross crop revenue per acre as a maximum.

High crop intensity (150% - 300%) areas also reduces the days the system is available for maintenance. Both cool and warm climate systems require scheduling a systematic and routine maintenance program.

2. Sustaining Production Measures-Control of Seepage to maintain maximum land area for production and to provide adequate water supply, control of weeds on systems to reduce infestation and loss of production, require water users to make beneficial use of irrigation water as a right to use water. This move is important to future development of irrigated lands to minimize negative impacts being circulated by anti-irrigation groups. Excessive water losses reduce operation efficiency and create accountability problems. Poor drainage characteristics of the soil, whether from within farm units or areas excluded from farms, creates the question of liability for districts as well as loss of productive lands. Maintaining all possible productive acres for assessment purposes requires policies defining the responsibility of the Board of Commissioners as well as the water users. A full understanding of the long range consequences is required before a policy is set which recognizes the causes of the drainage problems.

3. Financial Responsibility for O&M and Repayment of Capital Construction Costs

The financial obligation to maintain and repay capital costs, may vary depending on philosophy of governing entity or respective governments. Undoubtedly, the greater degree of federal participation will normally increase the annual fund requirement.

To involve the water users in contributing toward the costs of operating and maintaining and repayment of capital costs of a project not only develops a consciousness about costs but also a respect for the system.

4. Degree of System Efficiency

To provide an adequate water supply (demand) to land owners, two criteria must be met. One, is to have a sufficient water supply and the second, is to have facilities that can deliver the water. The amount of water delivered to a farm can be affected by available water supply and facility shortage. The responsibility of management is one of how best to reduce the harmful effects of a water supply or facility shortage.

(a) Water Supply

The decision is one of how much can be expended to bring in a supplemental supply of water, if it is available. Additional storage or trans-basin or inter-basin water transfers are costly.

(b) Facility Shortage

The controlling agenda is to seek ways of modernizing the conveyance system, reduce conveyance losses, and improve operation and maintenance. Generally, this is known as rehabilitation and betterment (R&B). Here again, management must decide who pays for the work and at what rate. When one considers all factors affecting system delivery efficiency such as soils, climate, crops, density of irrigable land per mile (KM) of system, operation and maintenance practices, evapotranspiration rates of growths along systems, an earth-lined system can vary from about 45% to 70% efficient. On-farm efficiency is the landowner's responsibility under normal conditions and is one of the major concerns in computing the overall efficiency of the project. If one considers crop irrigation requirement (CIP) as standard for efficiency, then on-farm efficiency under gravity irrigation does not exceed 35% and can be as low as 15%. For example, assume system efficiency at 64% and on-farm efficiency at 35%, then the overall efficiency is 0.64 X 0.35 or 22%. If the crop irrigation requirement is 1.25 acre-feet, then the demand is 5.7 acre-feet at point of diversion. If one can increase the overall efficiency from 22% to 28% then the demand can be lowered from 5.7 acre-feet to 4.5 acre-feet per acre, a 22% reduction. The degree of system efficiency becomes a key issue in determining the extent of maintenance of the system.

III. GOOD MANAGEMENT AND MAINTENANCE PROGRAMS ARE DEVELOPED TO INCLUDE:

A. Equitable Treatment of All Programs

1. Distribution of water
2. Development of SOP'S (Standing Operating Procedures) and Policy Manuals
3. Authorized programs carried out

B. Efficient Operation

1. Use of and control of water
2. Water conservation and water salvage
3. Effective use of personnel and equipment
4. Timely action on programs

C. Effective Programs are Established By:

1. Reducing emergency measures due to inadequate planning
2. Avoiding returning to the same job twice--do it right the first time
3. Carrying out programs which are cost effective

D. Economic Operation

1. Avoid unnecessary expenditures through detailed long range planning
2. Control purchases to reduce unnecessary expenditures for stores; it is desirable, where possible, to purchase equipment from the same manufacturer to minimize stocking of a large number of parts and supplies, material, and supplies

IV. PROJECT O&M ORGANIZATIONS - FACTORS AFFECTING COMPOSITION OF ORGANIZATION

A. Organizational design consideration tailored to size of area 5,000 acres to 100,000 acres (2,000 Ha - 40,000 Ha) projects can become too large for efficient management by being spread over large areas.

B. O&M authority should be delegated to field offices for effective administration.

C. Labor intensive versus mechanical operation determines the number of employees and percentage of labor to total budget.

D. Cropping intensities (80% - 300%).

E. Topography, geography, ownership patterns (accounts to handle) need to be considered in organizational structure.

F. Need for accessibility and all weather roads.

G. Sources of funding (whether subsidized or water user funded).

H. Availability of parts and services for equipment, materials, fuels, etc.

I. Availability of qualified personnel.

J. Lines of authority for governance policies should be set and management provided authority to carry out policies.

K. Use of joint boards of control for a more effective and efficient operation.

V. PROGRAMS

MAINTENANCE - Annual inspections as prescribed by USBR set pattern for maintenance programs (both routine and special)

A. Structure Maintenance should be divided into 3 phases: preventive, regular, and repair (including major or replacement)

1. Structures should be judged for serviceability, safety, and economy. A rapid identification system for canals, laterals, turnouts, structures, and spills is desirable for quick reaction to structure repair or operation delivery.

2. A guide for estimating annual maintenance expense based on useful life of the facility should be established. Average annual maintenance costs can vary widely depending on size, cost of project, system, and many other variables including updating cost to present day values. These maintenance costs could vary from 1/2 to 1% to 3% or the indexed costs of the project.

3. Maintenance programs and policies should include:

- (a) R.O.W. for access
- (b) Weed control
- (c) Roads, O&M and access for public and maintenance of project facilities
- (d) Equipment maintenance
  - (1) Parts and service availability
  - (2) Ownership justification - when is it advisable to rent rather than own
  - (3) Depreciation scheduling
  - (4) Inventory control

B. Types of Repair and Decisions Required

1. Emergency - maximum consideration for safe guarding system

2. Temporary - time and crop consideration - time of crop season

3. Permanent - time consideration - will crops be damaged?

4. Regular or routine - preventive maintenance is the key factor in developing maintenance programs

5. R&B - delayed maintenance

VI. BUDGETING - MANAGEMENT FUNCTION

A. Factors that affect budget preparation:

1. Economic life of system

2. Economic life of equipment and annual usage

   (a) Justification for ownership
   (b) Replacement cost of equipment

3. Personnel costs

   (a) Full and part time
   (b) Benefits - retirement, vacations, sick leave, insurance, unemployment, workman's compensation

4. Utilities - radio, telephone, heat, lights

5. Supplies - fuel, chemicals, construction materials

6. General expenses

7. Reserves - one-year's operation

8. Shops - storehouse - other manufacturing or assembly

9. Powerplants

10. Income

11. Assessment needs

VII. PERSONNEL

A. Office functions include management and staff - full time depending on size of project

B. Personnel - must be people who are qualified and care about the project and are interested in the work

1. Manager and assistant managers - engineers on larger projects or equivalent experience

2. Finance officers trained in finance procedures

3. Ditchriders (canal operators)

    (a) Mathematically inclined and can keep good records
    (b) Irrigation experience desirable
    (c) Public relation oriented
    (d) Responsible person who likes his work with sufficient authority to carry out policies
    (e) Willing to perform his duties

4. Mechanics - factory or technical training desirable

5. Operators - experienced and trained, can perform preventive maintenance and assist mechanics in repair of equipment

6. Supply clerk - adept at inventory control, purchasing techniques, knowledge of equipment parts and service

7. Others staff - laborers, etc., should be acquired as needed with qualification to perform needed work

VIII. CONCLUSION

Irrigation of arid land can only be successful when productivity is not impeded by lack of interest in maintaining the project at the maximum level.

A. Need for effective maintenance is essential to maintain continuous productivity.

B. An irrigation project of national utility is a work of benevolence and landowners need to establish integrity and confidence of the American people that Federal, State investment is justified.

C. Stabilization of socio-economic conditions - establishing communities across the country avoiding crop failures on large scale.

D. Long term benefits are achieved only through long range planning.

E. Project O&M people are the ones who make the project a success; they correct the construction errors and live with the people on the project. They should be the most qualified of all the people associated with the project. These are the people who need to carry out the approved programs.

EXHIBITS

Exhibit 1

AA - Example of budget to levy assessments showing actual expenses according to finance records, budget of year in operation, and budget for next year's operation

A - Breakdown of wages

A-1- Wages, benefits, and distribution to accounts

B- Status Equipment Reserve part of budget Exhibit AA

C- Anticipated construction debt payout schedule

Exhibit 2

Suggested need for breakdown of budget to help plan with greater accuracy for future budgets. Note inflated prices used. Please also note - not related to budget in Exhibit AA.

Exhibit 3

Conveyance feature envelope curves for estimated O&M costs per mile.

It is important to note these are only estimated based on limited data but can serve as a guideline for your budget estimates. Each project can develop its own curves through records kept over years.

Exhibit 4

Draft sample of Greenfields Irrigation District summary sheet for budget determination created to coincide with auditor's and accountant's request for financial records. Details of each summary group are broken down to list items involved in summary.

Exhibit 5

Numbering system for easy identification in operating and maintaining project turnouts as well as system as a whole.

RIVER IRRIGATION DISTRICT
1988 BUDGET FOR 1989
JANUARY 9, 1988

	1986 BUDGET 1987 EXPENDITURES **ACTUAL**	1987 BUDGET FOR 1988 OPERATION	1988 BUDGET FOR 1989 OPERATION
BUDGET SUMMARY:			
I. General Expenses............	$ 68,379.	$ 72,500.	$ 78,000.
II. Employee Services...........	592,997.	581,700.	590,400.
III. Equipment Operating Expense.	95,532.	98,500.	105.500.
IV. Equipment Replacement.......	52,500.	50,000.	55.000.
V. Stores......................	87,044.	115,000.	117,000.
VI. Utilities..................	10,597.	15,000.	15.000.
VII. Contracts.................	281,760.	138.000.	89.000.
VIII. Construction Debt Service...	405,000.	405.000.	405,000.
IX. Contingency................	-0-	22,200.	16.500.
	$1,593,809.	**$1,497,900.**	**$1,471,400.**
DEDUCTIONS:			
Grazing Revenue.....................	$ 52,633.	$ 50,000.	$ 50.000.
Water Service Contracts............	16.892.	15.000.	15,000.
Excess Water Charges..............	5.577.	12,000.	6.000.
Work for Others...................	136.492.	50.000.	50,000.
Interest on Investments:			
*Reg. & Spec. Res., $1,209,000 @ 7%	119,065.	96,000.	84,000.
*Equipment Reserve, $ 128,000 @ 5%	7,359.	9,400.	6,400.
*O&M Collections, $ 400,000 @ 5%	13,348.	25,000.	20,000.
*Construction Col. $ 115,000 @ 5%	12,586.	10,800.	5.800.
Penalty on Delinquent Accounts.....	18,151.	9.000.	10,000.
Fort Shaw Irrigation District......	3,952.	3.500.	4,000.
Construction Surplus Reduction..... -(See Exhibit D)-	51,305.	58,000.	61.000.
TOTAL DEDUCTIONS...	$ 437,360.	$ 338,700.	$ 312,200.
ASSESSMENT COMPUTATION:			
BUDGET LESS DEDUCTIONS......	**$1,156,449.**	**$1,159,200.**	**$1,159,200.**
Irrigable Acres Assessed.	79,608.	79,725.	79.725.
O&M Per Acre Assessment..	12.28	12.04	12.04
Const. Per Acre Assessment	2.26	2.50	2.50
Total Per Acre Assessment.	**$14.54**	**$14.54**	**$14.54**

I certify that the above budget was duly set by the Board of Commissioners at a Regular Meeting held January 9, 1988 at River Irrigation District office in Courtney, Texas.

Dated January 9, 1988

President

*Reserve value as of December 31, 1988

EXHIBIT 1-AA

G.I.D BUDGET FOR 1988

		'87 ACTUAL	'88 1987 BUDGET FOR 1988 OPERATION	'89 1988 BUDGET FOR 1989 OPERATION
I.	GENERAL EXPENSES:			
	Election Expenses	$ 253.	$ 400.	$ 400.
	Insurance & Bond Premium	37.257.	45,000.	45,000.
	Legal Fees	1.872.	4,000.	4.000.
	Auditor	2,800.	3,000.	3,200.
	Services, Commissioners	2,569.	3.000.	3.000.
	Travel, Commissioners	502.	1,000.	1,000.
	Employee Trav/Tuition/Per Diem	1,709.	1.000.	2,000.
	Office, Postage	1,605.	1.600.	1.800.
	Office, Telephone	3,375.	3,200.	3,600.
	Office, Supplies	3.211.	2,300.	3,000.
	Office, Equipment	5,918.	3,500.	3.500.
	Radio Repair	641.	1.000.	1,000.
	Water Organization Dues	1.000.	1.500.	1.500.
	Miscellaneous	5,677.	3,000.	5,000.
	TOTAL GENERAL EXPENSES	**$68,379.**	**$ 73,500.**	**$ 78,000.**
II.	EMPLOYEE SERVICES:			
	Salaries (See Exhibit A)	$ 474.464.	$ 459.300.	$ 464.800.
	PERS (6.4%)	29,806.	29.500.	29,800.
	Social Security (7.5%)	34,791.	34,500.	34,900.
	Workmans Comp. (5.0%)±	23,539.	25.300.	27,900.
	Unemployment Comp. (1.8%)±	8,721.	16,100.	13,900.
	Group Insurance (2.1%)±	10,034.	12.000.	14.400.
	Severence Pay (2.5%)±	11,642.	5.000.	4,700.
	TOTAL EMPLOYEE SERVICES	**592,997.**	**581,700.**	**590,400.**
III.	EQUIPMENT OPERATING EXPENSES:			
	Gasoline	$ 15,240.	$ 17,000.	$ 17,000.
	Diesel	20,036.	22.000.	22.000.
	Propane	2,722.	4,000.	2,000.
	Grease & Oil	3,861.	4.500.	4.500.
	Tires	8,216.	12,000.	12.000.
	Equipment Repair	36,052.	35,000.	40,000.
	Shop Equip., Tools, Supplies	9,405.	8,000.	8,000.
	TOTAL EQUIP. OPERATING EXP.	**$ 95,532.**	**102,500.**	**105,500.**
IV.	EQUIPMENT REPLACEMENT: (See Exhibit B)	$ 52,500.	$ 50,000.	$ 55,000.
V.	STORES:			
	Weed Chemicals	$ 34,763.	$ 20,000.	$ 32,000.
	Cement	3,264.	6,000.	5,000.
	Rebar	4,175.	10,000.	6,000.
	Pipe	20,401.	40,000.	40,000.
	Steel	5,480.	7,000.	7,000.
	Lumber	888.	5.000.	3,000.
	Coatings, Sealants, Admixtures	1,543.	2,000.	2,000.
	Hardware	866.	6,000.	4,000.
	Welding Supplies	2,343.	4,000.	3,000.
	Miscellaneous	13,321.	15,000.	15,000.
	TOTAL STORES	**87,044.**	**115,000.**	**117,000.**

Exhibit 1-AA contd

G.I.D BUDGET FOR 1988

		'87 ACTUAL	'88 1987 BUDGET FOR 1988 OPERATION	'89 1988 BUDGE FOR 1989 OPERATIC
VI.	UTILITIES:			
	O&M Headquarters	$ 7,280.	$ 10,000.	$ 10,000.
	Pumping Plants	3,317.	5,000.	5,000.
	TOTAL UTILITIES	**$ 10,597.**	**$ 15,000.**	**$ 15,000.**
VII.	CONTRACTS:			
	Concrete Lining	$ 58,235.	$ -0-	$ 20,000.
	Ready Mix Concrete	4,470.	5,000.	5,000.
	Canal Lining	215,035.	130,000.	60,000.
	Aerial Spraying	4,020.	3,000.	4,000.
	TOTAL CONTRACTS	**$ 281,760.**	**$ 138,000.**	**$ 89,000.**
VIII.	CONSTRUCTION DEBT SERVICE: (See Exhibit C)			
	Original Construction	$ 315,000.	$ 315,000.	$ 315,000.
	R&B Loan	90,000.	90,000.	90,000.
	CONSTRUCTION DEBT SERVICE	**$ 405,000.**	**$ 405,000.**	**$ 405,000.**
IX.	CONTINGENCY: (1-2%):	-0-	$ 22,100.	$ 16,500.

Exhibit 1-AA contd

EXHIBIT A

	'87 ACTUAL	'88 EXPECTED	'89 BUDGET
OFFICE Management	$ 37,577.	$ 37.500.	$ 38,600.
Finance	25.746.	25.700.	26,500.
Water-Land Records	22,318.	22,300.	23.000.
Reception - Computer - Secretarial	15.559.	16,700.	17,200.
MAINTENANCE FOREMAN (1)	19.247.	-0-	-0-
O&M FOREMAN (1)	27,361.	28.100.	28.900.
DAMTENDER (1)	23,399.	23,400.	24.100.
DITCHRIDERS (8)	22,838.	22,800.	23,500.
	22,603.	22,600.	23,300.
	22,400.	5,000.	5.200.
	22,314.	22.300.	23,000.
	22,138.	22.100	22.800.
	21,862.	21,900.	22,600.
	21,808.	21,800.	22,500.
	16.958.	22,200.	22,900.
OPERATORS (2)	20,057.	20,100.	20,700.
	15,369.	21,500.	22,100.
MAINTENANCE MEN (4)	20,408.	20,400.	21,000.
	15,292.	15,300.	15.800.
	8,570.	13,500.	13,900.
	7,597.	-0-	-0-
SHOP MECHANIC (1)	19,838.	19,800.	20.400.
SEASONAL VARIABLE	19,455.	26,000.	26,800.
	$ 474,465.	$ 451,000.	$ 464,800.

EXHIBIT A-1 WAGES AND BENEFITS

	1987 Wages Only	1989 Budget Wages and 27.022% Benefits	Adm.	Op.	Maint.	R. & B.
1. Manager	$37,577	$49,030	$24,515	$4,903	$9,806	$9,806
2. Secy.-Treas.	25,746	33,660	33,660			
3. Land-Water Records	22,318	29,215	11,686	17,529		
4. Receptionist-Clerk	15,559	21,847	14,567	3,640	1,820	1,820
5. Maintenance Foreman (1)	19,247		--	--	--	--
6. O & M Foreman (1)	27,361	36,709		22,025	7,342	7,342
7. Damtender (1)	23,399	30,612		18,367	12,245	
8. Ditchrider	22,838	29,850		17,910	5,970	5,970
9. Ditchrider	22,603	29,596		17,757	5,919	5,920
10. Ditchrider	22,400	6,605		6,605		
11. Ditchrider	22,314	29,215		17,529	5,843	5,843
12. Ditchrider	22,138	28,961		17,377	5,792	5,792
13. Ditchrider	21,862	28,707		17,224	11,483	
14. Ditchrider	21,808	28,580		17,148	11,432	
15. Ditchrider	16,958	29,088		17,453	11,635	
16. Operator	20,057	26,293			15,775	10,518
17. Operator	15,369	28,071			14,035	14,036
18. Maintenance Man	20,408	26,674			16,004	10,670
19. Maintenance Man	15,292	20,069			12,041	8,028
20. Maintenance Man	8,570	17,656			17,656	
21. Maintenance Man	7,597	--	--	--	--	--
22. Shop Foreman (1)	19,838	25,912			25,912	
23. Seasonal (Variable)	19,455	34,041			17,020	17,021
TOTAL:	$474,465	$590,391	$84,428	$195,467	$207,730	$102,766
80,000 Acres Cost/AC		$7.379	$1.055	$2.443	$2.596	$1.284
Wages expressed as % of Assessment ($14.54) = 50.7%			7.255%	16.801%	17.854%	8.830%

Exhibit A-1

EXHIBIT B

Equipment Reserve Status and Proposed Expenditures

December 31, 1987 Balance............	$	181.671.
1988 new 4x2 pickups (3).............		-29,900.
1988 new 4x4 pickup (1)..............		-11,500.
1988 used service truck (1 ton)......		- 4,000.
1988 sales of used pickups (7)......		+ 2,100.
1988 Cat 215 backhoe replacement....		-60,000.
1988 reserve reimbursement...........		+50.000.
	$	128,371.

EXHIBIT C

ANTICIPATED CONSTRUCTION DEBT PAYOUT

ORIGINAL CONSTRUCTION DEBT

YEAR	BEGINNING BALANCE	PAYMENT
1988	$1.847,206.	$315,000.
1989	1,532,206.	315,000.
1990	1,217.206.	315.000.
1991	902,206.	315,000.
1992	587.206	315,000.
1993	272,206.	272,206.
1994	-0-	

R&B DEBT

YEAR	BEGINNING BALANCE	PAYMENT
1988	$ 7.840,997.	$ 90,000.
1989	7.750,997.	90.000.
1990	7,660.997.	90,000.
1991	7.570,997.	90,000.
1992	7,480.997.	90,000.
1993	7.390,997.	132,794.
1994	7.258.203.	405.000.
1995	6.853.203.	405.000.
1996	6,448.203.	405.000.
1997	6.043.203.	405,000.
1998	5.638.203.	405.000.
1999	5.233.203.	405.000.
2000	4.828.203.	405,000.
2001	4,423,203.	405,000.
2002	4,018.203.	405.000.
2003	3,613.203.	405.000.
2004	3.208.203.	405.000.
2005	2.803,203.	405.000.
2006	2.398,203.	405,000.
2007	1,993,203.	405.000.
2008	1,588.203.	405,000.
2009	1.183,203.	405,000.
2010	778.203.	405.000.
2011	373,203.	373.203.

EXAMPLE

P=Propane 1.20 Gallon G=Gasoline 2.20 Gallon D=Diesel 2.20 Gallon		Est. Annual Use	Diesel & Gas Propane Gals.	2.20/gal. 1.20/gal. Cost	O&M	R&B
Equipment-Gas-Diesel (Cont)						
JD5010 Scraper @ 4gph	D	300 Hr	1,200	2,640.00	800.00	1,840.00
NW Dragline @ 3 gphr	D	400 Hr	1,200	2,640.00	2,640.00	
Lorain @ 3 gphr	D	400 Hr	1,200	2,640.00	2,640.00	
W-18 Loader @ 4 gphr	D	290 Hr	1,160	2,552.00	1,276.00	1,276.00
Trk-Trac Kenworth 5MP	D	5,000 Mi	1,000	2,200.00	1,100.00	1,100.00
JD670 Patrol 4 gphr	D	500 Hr	2,000	4,400.00	2,200.00	2,200.00
SUBTOTAL		86,500 Mi 5,900 Hr	G 6,309 D 31,640 P ------	$83,488.00	$40,847.00	$42,641.00
GRAND TOTAL		280,300 Mi 5,900 Hr	G=14,878 D=34,462 P= 7,317	32,732.00 75,816.00 8,780.00		
				$117,328.00	$66,172.00	$51,154.00

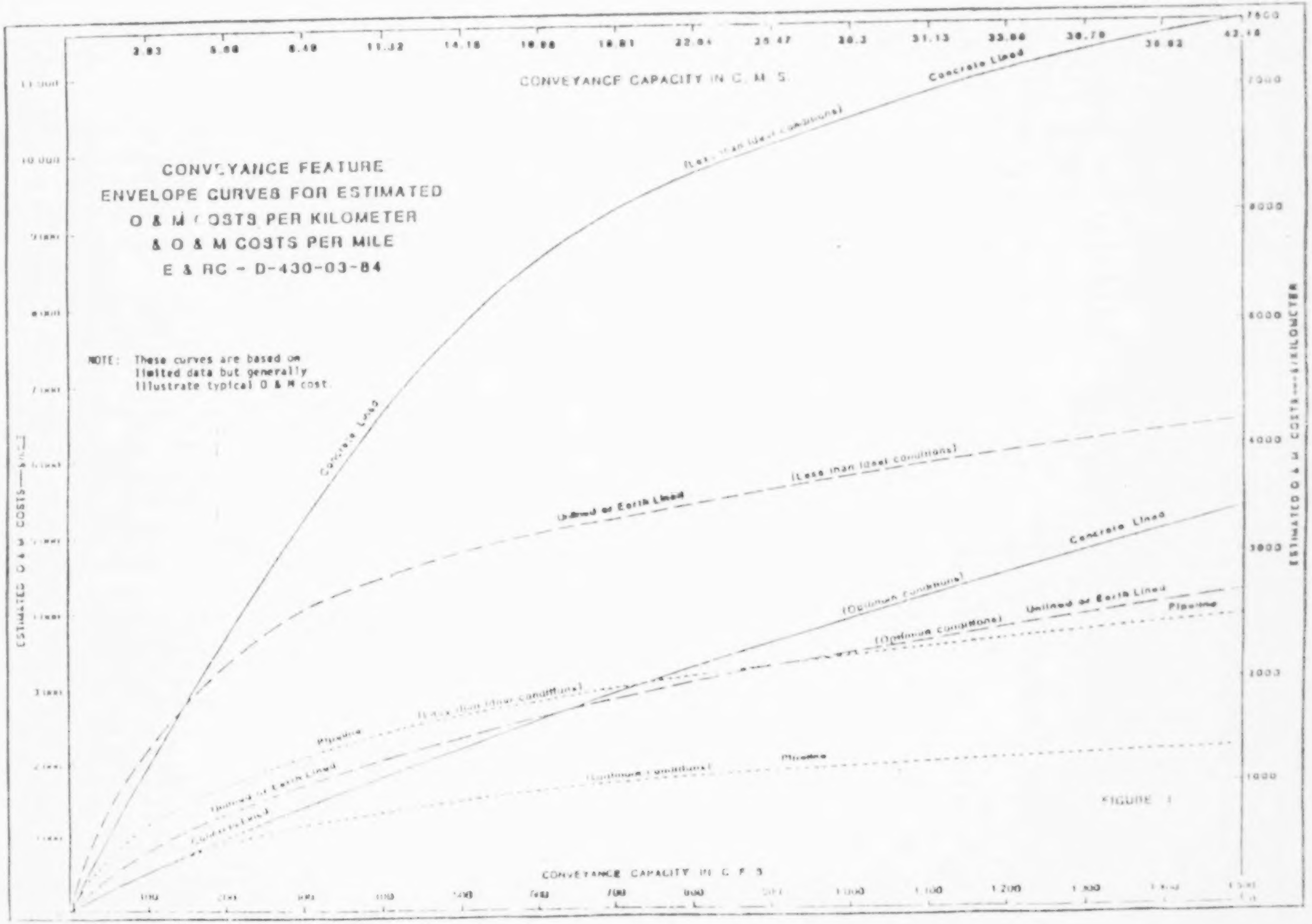
Fuel Oil-Filters-Grease		1980 Actual	Proposed 1982	O&M	R&B
Gas		15,334.47	32,732.00)		
Diesel		9,862.93	75,816.00)	66,174.00	51,154.00
Propane		4,153.28	8,780.00)		
Filters		1,743.95	2,500.00	1,410.00	1,090.00
Motor Oil, Hydraulic, Trans.		1,966.44	2,600.00	1,466.00	1,134.00
Grease		341.40	600.00	338.00	262.00
Misc. Washer, Cleaners			1,500.00	1,500.00	
SUBTOTAL		33,402.47	124,528.00	70,888.00	53,640.00
Tires					
Pickups New 40 @ 100	)		4,000.00	3,200.00	800.00
Recap 40 @ 50	)		2,000.00	2,000.00	
Truck Tires New 12 @ 250	)		3,000.00	3,000.00	
Recap 12 @ 100	)--	10,343.67	1,200.00	1,200.00	
Diesel Trucks New 10 @ 250	)		2,500.00		2,500.00
Recap 10 @ 100	)		1,000.00	1,000.00	
Hvy. Equip. New 4 @ 500	)		2,000.00	1,000.00	1,000.00
Recap 4 @ 300	)		1,200.00	1,200.00	
SUBTOTAL		10,343.67	16,900.00	12,600.00	4,300.00
Equipment Repair					
O&M 7% of 153,374.82		20,649.95	10,736.00	16,028.00	10,000.00
R&B 3% of 509,742.80			15,292.00		
SUBTOTAL		20,649.95	26,028.00	16,028.00	10,000.00
Maintenance of Machine Shop					
Utilities & Misc.		2,377.89	4,000.00	2,500.00	1,500.00
Materials			1,000.00	1,000.00	
Equipment			1,000.00	1,000.00	
Other Bldgs.			1,000.00	1,000.00	
SUBTOTAL		2,377.89	7,000.00	5,500.00	1,500.00

EXHIBIT 2

EXAMPLE

P=Propane G=Gasoline D=Diesel	1.20 Gallon 2.20 Gallon 2.20 Gallon		Est. Annual Use	Diesel & Gas Propane Gals.	2.20/gal. 1.20/gal. Cost	O&M	R&B
Ditchriders							
#1	8 MPG	G	14,000 Mi	1,750	3,850.00	3,850.00	
#2	10 MPG	P	12,000 Mi	1,200	1,440.00	1,440.00	
#3	12 MPG	G	9,800 Mi	817	1,797.00	1,797.00	
#4	9 MPG	G	11,000 Mi	1,222	2,688.00	2,000.00	688.00
#5	9 MPG	G	9,500 Mi	1,055	2,321.00	2,321.00	
#6	9 MPG	G	10,000 Mi	1,111	2,442.00	2,442.00	
#7	14 MPG	D	21,500 Mi	1,536	3,379.00	2,264.00	1,115.00
#8	14 MPG	D	9,000 Mi	643	1,415.00	1,415.00	
#9	14 MPG	D	9,000 Mi	643	1,415.00	1,415.00	
SUBTOTAL			105,800 Mi	G 5,955 D 2,822 P 1,200	$20,747.00	$18,944.00	$1,803.00
Supervisory							
Maintenance	8 MPG	P	16,000 Mi	2,000	2,400.00	2,160.00	240.00
Survey	10 MPG	P	7,000 Mi	700	840.00	340.00	500.00
Management	10 MPG	P	12,000 Mi	1,200	1,440.00	720.00	720.00
Foreman	11 MPG	G	6,500 Mi	591	1,300.00	390.00	910.00
Engineer	10 MPG	G	12,500 Mi	1,250	2,750.00		2,750.00
Operator	12 MPG	P	20,000 Mi	1,667	2,000.00	1,500.00	500.00
Operators							
#1	10 MPG	P	5,500 Mi	550	660.00	260.00	400.00
#2	11 MPG	G	4,000 Mi	364	801.00	561.00	240.00
#3	11 MPG	G	4,509 Mi	409	900.00	450.00	450.00
SUBTOTAL			88,000 Mi	G 2,614 D ----- P 6,117	$13,091.00	$6,381.00	$6,710.00
Equipment-Gas-Diesel							
A.C. Backhoe 1.5		G	150 Hr	225	495.00	495.00	
5-2T Trucks	6 MPG	G	18,500 Mi	3,084	6,785.00	4,750.00	2,035.00
Trk-Tractor Int	5 MPG	G	6,000 Mi	1,200	2,640.00	1,840.00	800.00
Other P.U.'s	8 MPG	G	12,000 Mi	1,500	3,300.00	2,310.00	990.00
Misc Equipment		G		300	600.00	330.00	330.00
Diesel Trucks (2)	5 MPG	D	45,000 Mi	9,000	19,800.00	6,000.00	13,800.00
Conc Mixer Truck		D	------				
Cat 225 @ 3.5 g.p.hr		D	1,000 Hr	3,500	7,700.00	2,310.00	5,390.00
Drott @ 4.00 g.p.hr		D	600 Hr	2,400	5,280.00	2,100.00	3,180.00
JCB @ 1.5 g.p.hr		D	400 Hr	600	1,320.00	1,000.00	320.00
D6D @ 6.0 g.p.hr		D	600 Hr	3,600	7,920.00	3,220.00	4,700.00
D-7 @ 3.0 g.p.hr (old)		D	60 Hr	180	396.00	396.00	
1150A @ 4.0 g.p.hr		D	600 Hr	2,400	5,280.00	3,780.00	1,500.00
JD350 Dozer @ 2 g.p.hr		D	100 Hr	200	440.00	340.00	100.00
JD860B Scraper @ 5gph		D	500 Hr	2,000	4,400.00	1,320.00	3,080.00

EXHIBIT 2 - contd


CONVEYANCE FEATURE
ENVELOPE CURVES FOR ESTIMATED
O & M COSTS PER KILOMETER
& O & M COSTS PER MILE
E & RC – D-430-03-84

NOTE: These curves are based on limited data but generally illustrate typical O & M cost.

FIGURE 1

EXHIBIT 3

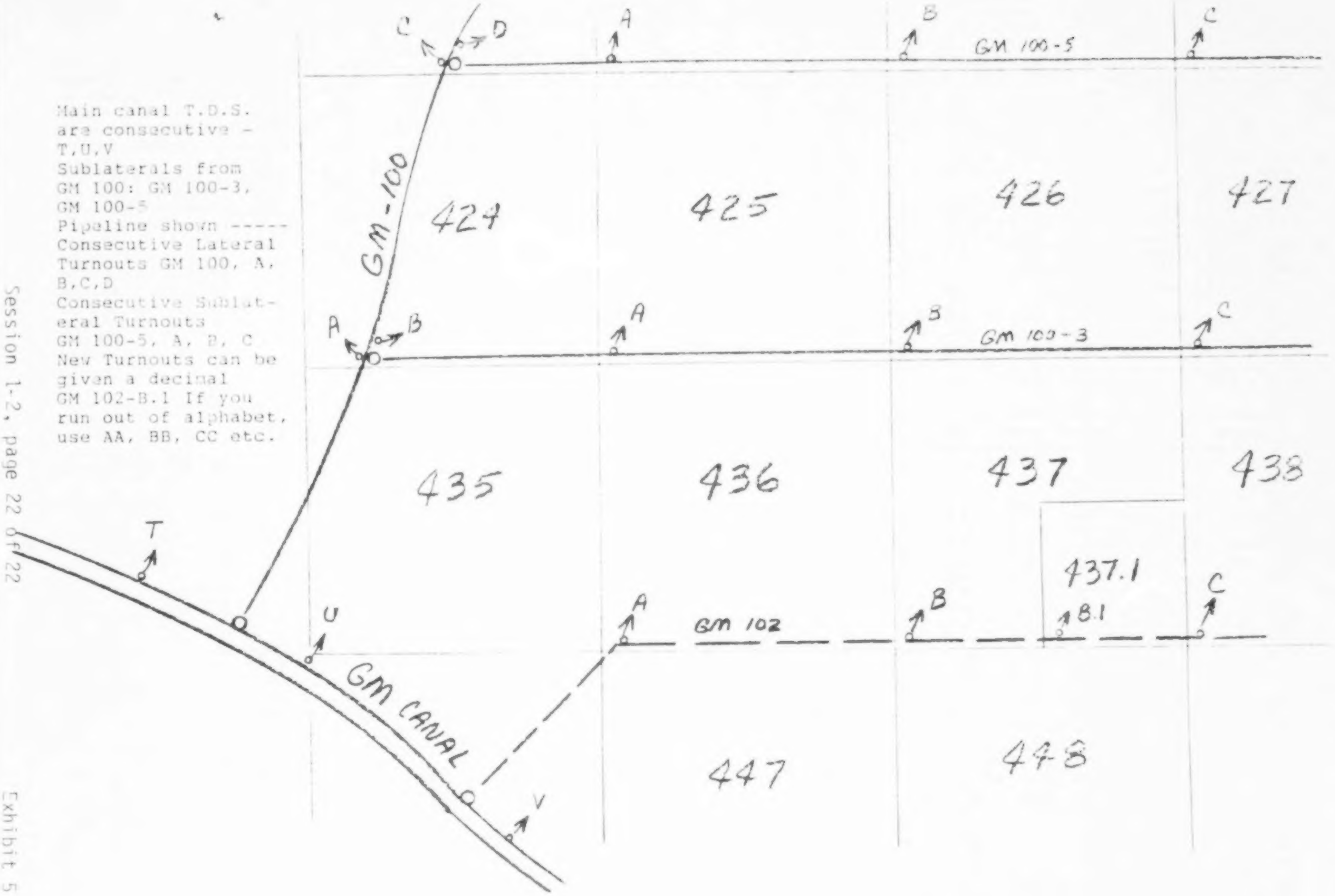
ENFIELDS IRRIGATION DISTRICT:
1989 BUDGET FOR 1990

ITEM	O&M FUND			U.S CONTRACT FUND			RESERVE FUND			EQUIPMENT FUND			TOTAL		
	'88 ACTUAL	'89 BUDGET	'90 BUDGET	'88 ACTUAL	'89 BUDGET	'90 BUDGET	'88 ACTUAL	'89 BUDGET	'90 BUDGET	'88 ACTUAL	'89 BUDGET	'90 BUDGET	'88 ACTUAL	'89 BUDGET	'90 BUDGET
BEGINNING FUND BALANCES:	$732	$0	$0	$176142	$123000	$65100	$1204307	$1192703	$1189603	$181672	$89015	$106915	$1562853	$1404718	$1361618
GENERAL EXPENSES:															
AUDITOR	$3431	$3200	$3500	$0	$0	$0	$0	$0	$0	$0	$0	$0	$3431	$3200	$3500
ELECTION	$330	$400	$400	$0	$0	$0	$0	$0	$0	$0	$0	$0	$330	$400	$400
INSURANCE & BONDS	$35515	$45000	$40000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$35515	$45000	$40000
LEGAL COUNSELING	$9807	$4000	$6000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$9807	$4000	$6000
OFFICE EQUIPMENT	$2719	$3500	$3500	$0	$0	$0	$0	$0	$0	$0	$0	$0	$2719	$3500	$3500
OFFICE SUPPLIES	$1746	$3000	$2000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$1746	$3000	$2000
POSTAGE	$2756	$1800	$3000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$2756	$1800	$3000
SERVICES, COMMISSIONERS	$2719	$3000	$3000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$2719	$3000	$3000
TRAVEL, MTGS., SCHOOLS	$2305	$3000	$2200	$0	$0	$0	$0	$0	$0	$0	$0	$0	$2305	$3000	$2200
UTILITIES, BUILD. & GROUNDS	$9645	$11000	$11000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$9645	$11000	$11000
UTILITIES, PUMPING PLANTS	$7851	$5000	$5000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$7851	$5000	$5000
UTILITIES, TELEPHONE	$3591	$3600	$3800	$0	$0	$0	$0	$0	$0	$0	$0	$0	$3591	$3600	$3800
WATER ORGANIZATION DUES	$1500	$1500	$1500	$0	$0	$0	$0	$0	$0	$0	$0	$0	$1500	$1500	$1500
MISCELLANEOUS	$4306	$5000	$5000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$4306	$5000	$5000
TOTAL GENERAL EXPENSE:	$88221	$93000	$89900	$0	$0	$0	$0	$0	$0	$0	$0	$0	$88221	$93000	$89900
EMPLOYEE SERVICES:															
INSURANCE, GROUP HEALTH	$8600	$14400	$9200	$0	$0	$0	$0	$0	$0	$0	$0	$0	$8600	$14400	$9200
INSURANCE/WORMANS COMP (5%)	$21979	$27900	$25100	$0	$0	$0	$0	$0	$0	$0	$0	$0	$21979	$27900	$25100
PERS (6.4%)	$29887	$29800	$32100	$0	$0	$0	$0	$0	$0	$0	$0	$0	$29887	$29800	$32100
SEVERENCE PAY (1.6%)	$0	$4700	$8000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$4700	$8000
SOCIAL SECURITY (7.5%)	$34942	$34900	$37600	$0	$0	$0	$0	$0	$0	$0	$0	$0	$34942	$34900	$37600
UNEMPLOYMENT COMP. (1.6%)	$7619	$13900	$8000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$7619	$13900	$8000
WAGES	$465273	$464800	$501700	$0	$0	$0	$0	$0	$0	$0	$0	$0	$465273	$464800	$501700
TOTAL EMPLOYEE SERVICES:	$568300	$590400	$621700	$0	$0	$0	$0	$0	$0	$0	$0	$0	$568300	$590400	$621700
EQUIPMENT EXPENSE:															
GASOLINE	$12000	$17000	$16000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$12000	$17000	$16000
DIESEL	$12055	$22000	$16000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$12055	$22000	$16000
PROPANE	$1935	$2000	$2000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$1935	$2000	$2000
GREASE & OIL	$4465	$4500	$4600	$0	$0	$0	$0	$0	$0	$0	$0	$0	$4465	$4500	$4600
TIRES	$11944	$12000	$12500	$0	$0	$0	$0	$0	$0	$0	$0	$0	$11944	$12000	$12500
EQUIPMENT REPAIR	$36666	$40000	$40000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$36666	$40000	$40000
SHOP EQUIP., TOOLS, SUPPLIES	$10679	$8000	$10000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$10679	$8000	$10000
EQUIPMENT RESERVE FUND	$65500	$55000	$65000	$0	$0	$0	$0	$0	$0	$65500	$55000	$65000	$65500	$55000	$65000
EQUIPMENT REPLACEMENT	$0	$0	$0	$0	$0	$0	$0	$0	$0	$158157	$37100	$48500	$158157	$37100	$48500
TOTAL EQUIPMENT EXPENSE:	$155271	$160500	$166100	$0	$0	$0	$0	$0	$0	$92657	($17900)	($16500)	$247928	$142600	$149600
STORES:															
CEMENT	$2570	$5000	$5500	$0	$0	$0	$0	$0	$0	$0	$0	$0	$2570	$5000	$5500
COATINGS/SEALANTS/ADMIXTURE	$1799	$2000	$2000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$1799	$2000	$2000

Exhibit 4

HARDWARE	$2065	$4000	$9500	$0	$0	$0	$0	$0	$0	$0	$0	$0	$2065	$4000	$9500
LUMBER	$1137	$3000	$3000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$1137	$3000	$3000
PIPE	$81227	$32900	$26000	$0	$0	$0	$0	$7100	$20000	$0	$0	$0	$81227	[illegible]	[illegible]
REBAR	$1111	$6000	$6000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$1111	$6000	$6000
STEEL	$4205	$7000	$7000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$4205	$7000	$7000
WEED CHEMICAL	$35132	$32000	$35000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$35132	$32000	$35000
WELDING SUPPLIES	$0	$3000	$3000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$3000	$3000
MISCELLANEOUS	$16973	$15000	$15000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$16973	$15000	$15000
TOTAL STORES:	$146189	$109900	$104500	$0	$0	$0	$0	$7100	$20000	$0	$0	$0	$146189	$117000	$126500
													$0		
CONTRACTS:													$0		
													$0		
CONCRETE LINING	$36078	$0	$0	$0	$0	$0	$0	$20000	$70000	$0	$0	$0	$36078	$20000	$70000
AERIAL SPRAYING	$5585	$4000	$4000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$5585	$4000	$4000
READY MIX CONCRETE	$6895	$5000	$7000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$6895	$5000	$7000
SRS LINING	$61500	$0	$0	$0	$0	$0	$115000	$60000	$0	$0	$0	$0	$176500	$60000	$0
MISCELLANEOUS	$11045	$0	$0				$0	$0	$0	$0	$0	$0	$11045	$0	$0
TOTAL CONTRACTS:	$121103	$9000	$11000	$0	$0	$0	$115000	$80000	$70000	$0	$0	$0	$236103	$89000	$81000
DEBT SERVICES:											$0	$0			
											$0	$0			
ORIGINAL CONSTRUCTION	$0	$0	$0	$318213	$315000	$315000	$0	$0	$0	$0	$0	$0	$318213	$315000	$315000
R&B LOAN	$45000	$90000	$85200	$0	$0	$0	$0	$0	$0	$0	$0	$0	$45000	$90000	$85200
TOTAL DEBT SERVICE:	$45000	$90000	$85200	$318213	$315000	$315000	$0	$0	$0	$0	$0	$0	$363213	$405000	$400200
CONTINGENCY:	$0	$16500	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$16500	$0
TOTAL BUDGET:	$1124084	$1069300	$1080400	$318213	$315000	$315000	$115000	$87100	$90000	$43500	$53000	$45000	$1649954	$1453500	$1468900
BUDGET DEDUCTIONS:															
EXCESS WATER FEES	$43461	$6000	$6000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$43461	$6000	$6000
FT. SHAW IRRIGATION DIST.	$0	$4000	$2500	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$4000	$2500
GRAZING REVENUE	$0	$0	$0	$55045	$50000	$50000	$0	$0	$0	$0	$0	$0	$55045	$50000	$50000
INTEREST ON INVESTMENTS:															
REGULAR & SPECIAL RESERVES	$0	$0	$0	$0	$0	$0	$103396	$84000	$90000				$103396	$84000	$90000
EQUIPMENT RESERVES	$0	$6400	$5000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$6400	$5000
O&M COLLECTIONS	$17102	$20000	$12000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$17102	$20000	$12000
CONSTRUCTION COLLECTIONS	$0	$0	$0	$7966	$5800	$5000	$0	$0	$0	$0	$0	$0	$7966	$5800	$5000
PENALTY+INTEREST ON DELINQ.	$14398	$8000	$8000	$5083	$2000	$2000	$0	$0	$0	$0	$0	$0	$19481	$10000	$10000
WATER SERVICE CONTRACT FEES	$16541	$15000	$15000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$16541	$15000	$15000
SALES - SERVICE & MATERIAL	$72302	$50000	$60000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$72302	$50000	$60000
TOTAL BUDGET DEDUCTIONS:	$163804	$109400	$108500	$68094	$57800	$57000	$103396	$84000	$90000	$0	$0	$0	$333294	$231200	$255500
NET BUDGET REQUIREMENT:	$960744	$959900	$971900	$199390	$199500	$212900	$0	$0	$0	$0	$0	$0	$1160944	$1159200	$1184000
ACRES ASSESSED	79796	79725	79835	79729	79725	79835	79729	79725	79835	79729	79725	79835	79729	79725	79835
REQUIRED ASSESSMENT PER ACRE:	$12.04	$12.04	$12.17	$2.50	$2.50	$2.67	$0.00	$0.00	$0.00	$0.00	$0.00	$0.00	$14.56	$14.54	$14.83
FUND BALANCES:	$1680	$0	$0	$122826	$65100	$20000	$1192703	$1189603	$1009603	$89015	$106915	$122415	$1406254	$1261618	$1152318

Exhibit 5


GM-100
GM 100-5
GM 100-3
GM 102
GM CANAL
A
B
C
D
T
U
V
B.1
424
425
426
427
435
436
437
437.1
438
447
448

Main canal T.D.S. are consecutive - T,U,V
Sublaterals from GM 100: GM 100-3, GM 100-5
Pipeline shown -----
Consecutive Lateral Turnouts GM 100, A, B,C,D
Consecutive Sublateral Turnouts GM 100-5, A, B, C
New Turnouts can be given a decimal GM 102-B.1 If you run out of alphabet, use AA, BB, CC etc.

Water Systems Operation and Maintenance Workshop - 1990

WATER SYSTEMS MANAGEMENT, PART 1
CANAL SYSTEMS AUTOMATION

Session 2-1

Dave Rogers and Scott Coulson

The Bureau of Reclamation is producing a new technical publication, the Canal Systems Automation Manual, written by Dave Rogers, Clark Buyalski, Dave Ehler, Hank Falvey, and Ed Serfozo. Part 1 of this manual, "Canal Automation Fundamentals," should be available in 1990. Excerpts from a draft of the Canal Systems Automation Manual are included in the session notes below. Additional material written by Scott Coulson completes the session notes.

Introduction

In recent years, the rapid advancement of automatic control equipment has greatly expanded the field of water system operation and control. Automation for water storage and conveyance systems can improve service to water users, increase efficiency and flexibility of operation, reduce the cost of construction and operation, and provide more responsive reaction to emergencies. The term **"automation"** can be defined as follows:

> A procedure or method used to control a water system by mechanical or electronic equipment which takes the place of human observation, effort, and decision.

Automating a water system is therefore the implementation of a control system that includes automatic monitoring and/or control equipment. A **control system** is an arrangement of electronic, electrical, and mechanical components that will command or direct the regulation of a canal system. In addition to equipment (hardware), control systems also include some form of programmed logic (software).

Bureau of Reclamation water use studies have shown that an average of only 44 percent of the water delivered to irrigators' fields on Bureau projects is stored in the root zone for beneficial consumptive use by crops. There is a real need for improvement of the management and utilization of water in irrigation operations. Application of modern automation technology can markedly improve the efficiency of project and farm irrigation systems and the In of conveyance and distribution systems. Improved operation will increase crop yields and make more water available for municipal, industrial, and recreational uses. The operation, control, and automation of canal systems is therefore an important topic for today's planners, designers, operators, and managers of irrigation water projects.

Canal Automation Needs

Recognizing the need for water system automation is essential. New principles of water distribution, application, and scheduling are being developed and implemented. These modern irrigation practices strain the capabilities of the conventional method of canal operation. The canal system needs to respond quickly with greater flexibility to achieve optimum water transfer efficiency.

In many newer canal systems, turnouts from the canal divert water into automatic pipe distribution systems. These distribution systems can be operated essentially on demand, limited only by the capability of the main canal system to respond. Pumps and valves are subject to power and equipment failure. When a canalside pumping plant shuts down, the entire diversion is rejected back into the main canal. The excess flow could propagate into the lower canal pools, causing undesirable operation of the wasteways or flooding. Conversely, a sudden or unforeseen increase in flow diverted at an automatic pump turnout could cause serious shortages of water to the downstream water users.

It is possible for the conventional canal operation to satisfy operational requirements through the application of better methods to predict water schedules. However, precise scheduling is difficult to implement on a total system basis; it is impossible to predict or anticipate some of the causes of turnout variation such as power and equipment failures.

The main limitation of a conventional canal system operation is the number of visits per day a ditchrider can make to the canalside turnout. Usually, the ditchrider only has enough time to make one or two trips down the canal system during the work day. Many canal systems do not provide regular ditchrider coverage during the nighttime hours. A ditchrider usually is available on a standby basis in case of emergency. It is absolutely necessary for the water user to adhere to a constant delivery according to the schedule. Any deviation from the schedule may affect other water users on the main canal system. Normally, water diversions to the canalside turnout are scheduled for a change of flow once daily.

On some canal systems, operation has simply become too difficult to be operated in the conventional manner. Frequent visits to the canal check gate structures are required by the ditchrider to accommodate significant variations in the daily canalside turnout diversion rate. The diversion rate may also deviate significantly from the scheduled daily rate. To provide efficient service to these turnouts, it probably is necessary to reduce the ditchrider's canal length coverage (circuit). Also, more ditchriders are required on a 24-hour basis to be available immediately to keep the canal system balanced and avoid large mismatches between the supply and demand. Even with increased ditchrider coverage, abnormal operations caused by sudden and unannounced changes in demand often cannot be avoided.

Purpose and Benefits of Canal Automation

The primary purpose of automation is to upgrade a canal operation. A canal operation is enhanced by installing a practical and modern control system to improve the water transfer efficiency. Many of the limitations inherent in the conventional method of operation can be overcome. Local automatic control, supervisory control, or a combination can provide the following advantages compared to the conventional method:

- Transfer of water rapidly (small or large quantities) on a frequent basis

- Achieve operational flexibility when simultaneous information on the entire system is known

- Adjustments in canal flow can be made to accommodate variances in canalside turnout diversions on a daily or hourly basis

- Immediate response to sudden and unannounced variances in canalside diversion or to storm runoff flooding into the canal prism

The advantages inherent in canal automation are appealing. A control system can be designed to ensure coupling of the canalside turnout diversions and the canal inlet supply. The canal system can respond to abrupt and unscheduled changes in turnout diversion and immediately initiate corrective action. The action reduces the necessity for excess flow and minimizes the occurrences of shortages in the lower canal pools. A nearly optimum transfer of water can be attained.

The objective of water system automation is to realize the benefits associated with implementing a practical control system. These benefits are:

- Better service to the water user
- Efficient transfer of water
- Reduction in operating costs

The water user is the main beneficiary. The user receives a scheduled quantity of water at the time specified without the limitations imposed by the conventional method of operation. The accompanying economic benefits and better service reward the water user. The monetary dollar value is not easily defined because of resulting intangible benefits. However, by providing the customer with improved service, the watermaster would certainly receive fewer complaints throughout the season.

Improved canal operations will result in economic benefits if the wasting of water is reduced or eliminated. Projects having to schedule power for relift pumping could realize a substantial economic savings by not pumping unnecessary surplus flows. Significant savings could also result in lower maintenance costs associated with canal lining and embankment failures. Excessive pressure changes on the canal lining and high water levels above the canal freeboard, which cause failures, can virtually be eliminated in the canal operation.

The reduction in O&M (Operation and Maintenance) costs associated with the installation of a control system constitutes the main tangible economic savings. Fewer ditchriders will be needed to operate the canal system after the control system is placed in operation. However, even with automation, minimum ditchrider coverage of the canal system will still be needed. The canal system requires daily patrol (as a safety policy) for observing the facilities to be aware of encroachment, to deal with vandalism, and to act on other potential hazards.

Automation increases the need for additional higher technical expertise for software and electronics work. Technology will be needed for applying general principles to solve control system development, implementation, and maintenance problems. The development and implementation work would probably be accomplished as part of the initial specifications contract and be considered as a capital cost of the project. However, skilled technicians will be required to perform maintenance. As a general rule, a cost reduction of two ditchriders will offset the cost of adding one electronic or computer software technician.

The water project will benefit even if the tangible operating cost reduction just offsets the cost of implementing a new control system. The intangible benefits associated with improved water service and water transfer will be significant and worth the endeavor.

The decision to implement a control system should not be justified solely on tangible economic benefits. Canal operations vary from one canal to the other depending on the type of inlet and outlet facilities, the flow capacity, and the nature of the service demands. Before a cost/benefit study can be accomplished, the operational and control requirements particular to a given canal system must be identified. Proper identification of requirements and needs will lead to the selection of either local automatic and/or supervisory control or to a continued operation by the conventional method.

CHAPTER 2. OPERATIONS

A. INTRODUCTION TO CANAL OPERATIONS

2-1. Principles of Operation

a. Definition - The canal system operation transfers water from its source(s) to one or more points of diversion downstream. Operations deal with the movement and behavior of the water in a canal system, and are dependent on the principles of open channel hydraulics. The primary function of operations is to manage the changes in flow and depth throughout the canal system. The term "operations" refers to the hydraulic reaction in the canal pools which results from control actions.

b. Objectives - From a most basic standpoint, canal systems are built for the purpose of transporting water from one place to another. The most prevalent reason for this transfer of water is the irrigation of farmlands, with the objective of helping farmers to increase crop yields. Canals also provide water for M&I (municipal and industrial) use, carry storm and snowmelt runoff to natural drainage channels, collect water from several independent sources into a single supply, convey water used in the generation of electrical power, and supply water for fish, wildlife, and recreation.

The objective of building and operating a canal system is to serve the above purposes as efficiently and economically as possible. In order to do this, canal operations should be tailored for each canal system to meet that system's specific requirements. But the requirements imposed on canal systems are steadily becoming more complex. Modern irrigation practices require greater delivery flexibility than in the past. The higher cost of water, power, and construction decreases the feasibility of wasting water from canals or building oversized canals which contain additional in-channel storage. The use of intermediate storage and wasteways is also being curtailed by environmental concerns. Because of these and other factors, there is an increasing need for canal systems to respond quickly to flow changes. The overall canal operation must be improved to provide this capability.

Automation of a canal system should not be thought of as an end, but rather as a means to accomplish the desired canal system operations. The true goal should be to achieve the most efficient and beneficial operation possible. Expanded control system capabilities is one method of economically reaching this goal.

c. Balanced pool operation - As defined in Chapter 1, the length of canal between check structures is called a canal pool. (Occasionally, an in-line pumping plant may serve as a pool boundary instead of a check structure.) When the total flow out of the canal pool equals the total inflow, the pool operation is balanced. Referring to figure 2-1, a balanced pool operation exists when:

$$Q_{C1} = Q_{TO} + Q_{C2}$$

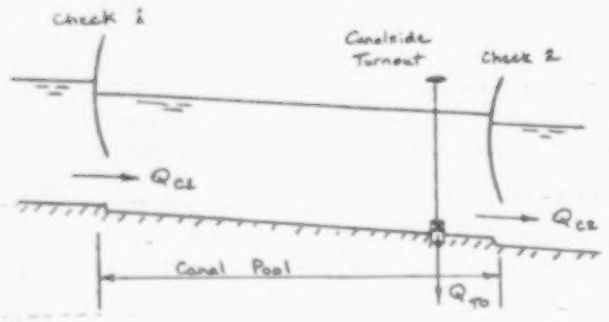

Check 1
Canalside Turnout
Check 2
Q_{c1}
Q_{c2}
Canal Pool
Q_{TO}

Figure 2-1. - Balanced Pool Operation

d. Conventional operation - Canals have traditionally been sized to convey the maximum design flow, with freeboard added as a safety factor against excess water depths. Canals which were designed in this way have usually been operated in the "conventional" manner. Figure 2-2 illustrates some of the characteristics of conventional operations.

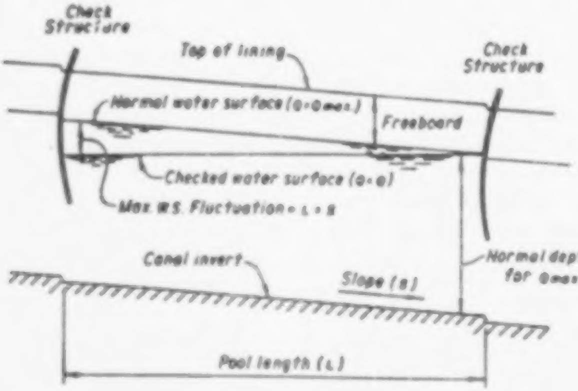

Check Structure
Top of lining
Check Structure
Normal water surface (Q=Qmax)
Freeboard
Checked water surface (Q=0)
Max W.S. Fluctuation = L × S
Canal invert
Slope (S)
Normal depth for Qmax
Pool length (L)

Figure 2-2. - Conventional Canal Operation

An important characteristic of conventional operation is the attempt to maintain a constant water depth on the upstream side of the check structures. Within each canal pool, the water surface profile essentially pivots about this constant depth at the downstream end of the pool when the flow changes.

Conventional operation has had such widespread use for so many years because:

1. Construction costs can be minimized by sizing the canal prism only large enough to convey the maximum steady flow.

2. The canal system operation can be successfully accomplished using local manual control.

e. Supply and demand - Canal operation concepts were defined in Chapter 1 based on whether the water schedule priorities are associated with supply or demand. Supply refers to the quantity of water which is available or being provided from an upstream source. The upstream operation concept applies to canal systems which are primarily supply-oriented. A supply-oriented system, therefore, must be operated to satisfy upstream requirements. One example of this would be a storm drainage system.

Demand refers to the water which is needed or being delivered at points downstream in the canal system. The downstream operation concept applies to canal systems which are primarily demand-oriented. These systems must respond to downstream conditions, and the water supplied from upstream is a function of downstream demand. A municipal water system is an example of a demand-oriented system. Water users at the downstream ends of the system take water without restriction, and the operation responds only to the downstream conditions.

f. Anticipation vs. reaction - Some canal systems only react to circumstances after they happen, while others are operated by anticipating situations in advance. Automatic controls are purely reactive, responding to present or past information. However, prediction is often used in manually controlled systems, and it can be used in automated systems which include manual input.

The most common application of canal operation using prediction is scheduled delivery. The scheduling of operations is based on water orders turned in by water users in advance of the actual delivery. Anticipation can also be based on historical data, daily or weekly trends, meteorological information, or agricultural data such as soil moisture content.

2-2. Canal Systems

As discussed in Chapter 1, a canal system can be classified according to its principal objectives as a delivery system, collector system, or connector system. There will be certain characteristics of each of these types of canal systems which affect operations. In addition to the different priorities associated with supply and demand discussed above, there are physical differences which need to be considered.

A delivery system has a large capacity at the upstream end which gradually decreases to a small capacity at the downstream end, sometimes branching into successively smaller lateral canals. Water is turned out at various points along the length of the canal as it tapers to the final turnouts at the tail end(s). This decrease in capacity amplifies the effects of a flow imbalance as it moves downstream.

Consider, for example, a canal with a 20 m^3/s capacity at the upstream end which tapers to a capacity of 4 m^3/s in the downstream pools. If there is a 1 m^3/s mismatch in flow at the upstream end, this is only a 5 percent error. This amount of error may be quite normal and acceptable, creating no significant problems in the upper pools. But as this 1 m^3/s mismatch is passed downstream, it will become quite significant in the downstream pools, where it will be a 25 percent flow error.

Water users located at the downstream end of delivery systems, referred to as tailenders, will often suffer from too much or too little water. Normally, surplus water must be supplied at the headworks to prevent shortages of water to tailenders. The surplus is often wasted at the downstream end of the system. Typical waste in a conventionally operated canal system is about 5-10 percent of the total inflow.

This problem stems from combining the downstream operation concept (demand-oriented) with the upstream control concept. As stated in Chapter 1, this is an inefficient combination. When using the upstream control concept, a large portion of the flow discrepancies tend to get passed downstream. These problems end up at the tail end of the system, where it is even more difficult to deal with them.

Delivery systems will operate more efficiently with a downstream control concept. Flow imbalance errors will be passed in the upstream direction, into successively larger pools. The effects of a flow imbalance will be attenuated as it moves upstream.

The same reasoning can be applied to the exact opposite situations for collector systems. Since collector systems tend to increase in capacity as you move downstream, operations will be most efficient when the upstream control concept is applied.

Connector systems will be neutral to these concepts, since the capacity should remain constant throughout the canal length. For a connector system, the source of the majority of the flow changes should determine which control concept is to be used. If flow changes generally originate at the upstream end, then the upstream control concept will work best, and vice versa.

The above discussion is particularly important when automatic control is involved. It will help to identify the types of automatic control systems which best suit a particular canal. In general, a canal system's operations must be clearly defined before automatic control can be successfully applied. This will make some canals better candidates for automation than others.

Physical characteristics also affect the potential for successful automation. An example is a steep channel slope. A canal with a steep bottom slope and/or many vertical drops in the invert will be much more difficult to automate than one with gentle slopes. Whenever there is supercritical flow, the upstream flow is no longer influenced by downstream conditions. This will prevent typical depth-based automatic controllers from working effectively.

The amount of storage within the canal prism is another important physical characteristic which influences canal operation and automation. Flow changes are typically accompanied by changes in the volume of water stored in the canal, because there will be different water surface profiles at different flow rates. These principles will be further developed later in this chapter.

2-3. Water Schedule

The water schedule for a canal system is the flow versus time relationship This can range from nearly constant flow for a canal with little flow change to a complex schedule of flow rates which change frequently at numerous points throughout a large canal system. Every canal will have a water schedule, but there are a variety of ways to predict, create, and implement the schedule. The water schedule is usually predicted or arranged in advance and then adjusted during implementation. The amount of flexibility to adjust the water schedule can vary widely from one canal to the next.

Schedules are often addressed in conjunction with delivery. Delivery refers to the release of water from the canal system to water users. This release usually occurs at some type of turnout structure. A delivery schedule is the quantity of water to be transferred to a user, as a function of time. Scheduled delivery is one type of delivery concept. Delivery concepts are discussed in section C of this chapter.

A conventionally operated canal will be used as an example to further explain the water schedule. With the conventional method, the water schedule specifies:

(1) the quantity of water to be transferred downstream,

(2) the location where specific quantities are to be diverted,

(3) the time of the diversion.

The development of the water schedule typically begins at least one day prior to the actual demand requirement. Individual users submit water orders to their ditchrider or to their district headquarters. The water district compiles the individual water orders for the distribution system associated with each canalside turnout from the main canal system.

The water district will sometimes arrange modifications to the individual water orders when it is not feasible to meet all the requests exactly. These arrangements will usually involve cutting back all of the individual orders by a certain percentage when there is not enough water to go around. It is also possible to change the timing of deliveries, to avoid large flow changes in the canals.

After all the individual orders are collected, the water districts submit the new canalside turnout orders to the watermaster of the main canal system. The watermaster then accumulates the canalside turnout orders by canal pool. The flow requirements for each canal pool are algebraically summed beginning at the most downstream pool, as shown in figure 2-3.

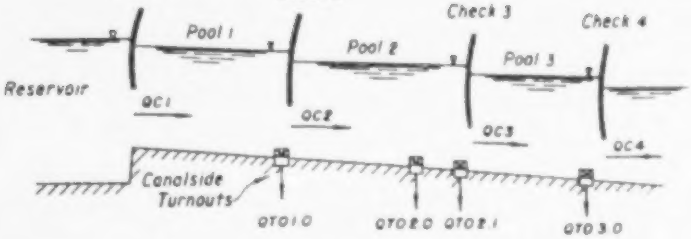


Figure 2-3. - Developing a Water Schedule

The canal flow requirement by canal check would be:

QC3 = QTO3.0 + QC4
QC2 = QTO2.0 + QTO2.1 + QC3
QC1 = QTO1.0 + QC2

The algebraic summation of the canal pool flow schedule progresses upstream to the canal headworks, Check 1. The water schedule for the canal system is now established for the next day. The schedule has the total input flow required at the headworks, all of the canalside turnout flow diversions, and the flow required at each successive canal check gate downstream to meet the turnout flow changes enroute.

Some canal systems operate with delivery of water to the users "on-demand," and the water schedule is not arranged in advance. In these cases, the schedule can often be predicted in advance based on typical water use patterns and any other known circumstances which will influence water use. The water schedule will then need to be adjusted in a "real-time" environment, in reaction to changing demands.

2-4. Water Transfer Procedure

In a conventional canal system, the ditchrider adjusts the check gates to satisfy the water schedule. The ditchrider must first determine the new canal check gate openings required for the flow changes. An experienced ditchrider is able to predict the relationship between gate opening and flow with reasonable accuracy. The ditchrider usually begins to implement the new water schedule early in the morning, by changing the flow at the head of the canal. The Check 1 gate opening is adjusted to obtain the new total flow, QC1. (S)He will then follow the flow change downstream, making the required adjustments at canalside turnouts and the successive check gates in sequence.

The check gates will usually be adjusted to maintain a desired depth on the upstream side of the check structure. This is the upstream control concept. This helps to achieve steady flows through gravity turnouts which are located just upstream of the check structure. As the flow change progresses downstream through each pool of the canal, the ditchrider will attempt to achieve a balanced pool operation.

Figure 2-4 shows a flow increase progressing downstream through the pool. When this flow change reaches the turnout, the turnout flow will be adjusted to the scheduled value. The difference between the flow change from the upstream check, QC_n, and the turnout flow change, QTO_n, will then proceed past the turnout to the next downstream check gate, Check n+1.

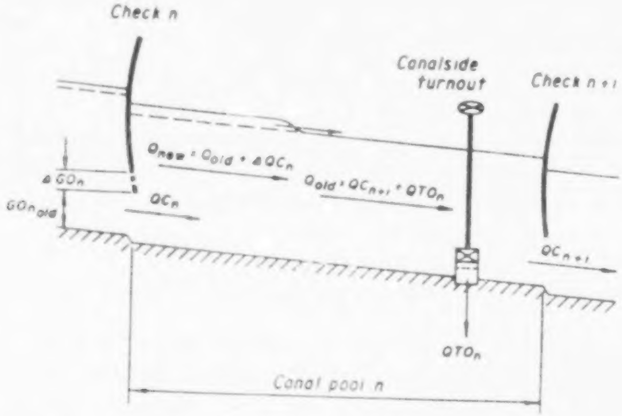

Check n
Canalside turnout
Check n+1
$Q_{new} = Q_{old} + \Delta QC_n$
ΔGO_n
$GO_{n_{old}}$
QC_n
$Q_{old} = QC_{n+1} + QTO_n$
QC_{n+1}
QTO_n
Canal pool n

Figure 2-4. - Flow Increase Progressing Downstream

When the flow change arrives at Check n+1, the check gate opening is adjusted to transfer the flow mismatch (between the upstream canal pool inflow and turnout outflow) into the next downstream canal pool. If Check n+1 is properly adjusted, the pool inflow and outflow will be balanced and the depth on the upstream side of Check n+1, YU, will remain relatively constant. A new steady-state flow is established, as shown in figure 2-5.

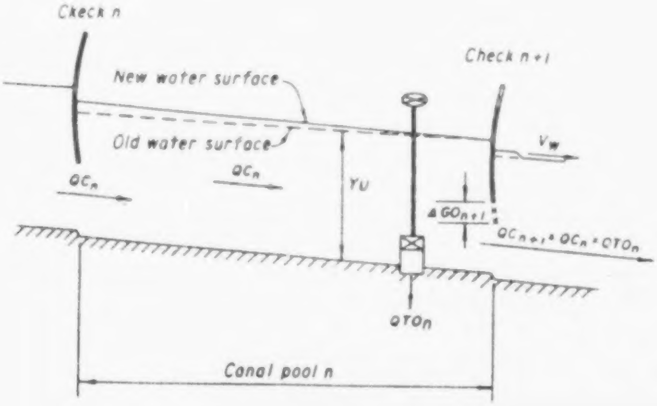

Ckeck n
Check n+1
New water surface
Old water surface
V_w
QC_n
QC_n
YU
ΔGO_{n+1}
$QC_{n+1} = QC_n - QTO_n$
QTO_n
Canal pool n

Figure 2-5. - Downstream Gate Adjustment to Transfer Flow Change

The sequence of making flow changes and balancing the canal pool operations then continues downstream until all of the turnout flow changes have been made according to the water schedule. It is difficult for a ditchrider to correct flow imbalances with just a single gate adjustment at each check structure. In a real canal, it isn't as easy as it appears in this idealized example.) Therefore, only one trip down the canal by the ditchrider is usually not sufficient. Typically, subsequent gate adjustments are required to balance the flow conditions.

The procedure described above is one example to help illustrate some of the details of transferring water through a canal system. It is a typical procedure for the great numbers of canals which use the conventional method. Different water transfer procedures will be used for canal systems which are operated and controlled with different methods.

C. DELIVERY CONCEPTS

2-8. General

The degree of freedom that water users have related to the time and quantity of the water they receive, defines the delivery concept:

- Rotation - Users share a constant water supply, cooperating with other water users to establish the time and quantity of delivery.
- Scheduled - Advanced notification of delivery time and quantity is required, limited by the contractual water allotment on a daily, monthly, and yearly basis.
- Demand - Unrestricted use of the available water supply, limited only by a maximum flow rate and total allotment.

The delivery concept establishes important operating criteria for the main canal system. The flow response characteristics in the canal system are different for each delivery concept. The rotation delivery concept offers the easiest canal operation, since flow changes at the canalside turnouts are infrequent and water level fluctuations in the canal prism are minimal. There will be more flow changes with the scheduled delivery concept, which increases the complexity of canal operations. The predictability of these scheduled flow changes helps to minimize the resulting water level fluctuations. For the demand delivery concept, the main canal needs to respond immediately to unannounced and frequent flow changes at the canalside turnouts. Unannounced flow changes, small or large, may cause relatively large water level fluctuations in the canal prism.

2-9. Rotation Delivery

The rotation delivery concept rotates a relatively constant canal turnout flow among a group of water users. The constant turnout flow is divided by time and quantity so that each individual water user receives a share of the total diversion. Rotation delivery probably causes the most inconvenience to the individual water users. They must cooperate fully with other water users so that everyone gets his share of the constant available water supply. Usually, they must rotate the specific timing of their irrigation schedule in order to improve the efficiency of water application to their crops. Last-minute changes in the schedule are simply not allowed, because they would have an adverse effect on the schedule of several other water users. It is a take or lose proposition. Severe economic losses could occur if the water user is not proficient or cooperative.

Even when full proficiency and cooperation are achieved, the rotation delivery concept causes inefficient irrigation water use. The size of a crop to be irrigated will be based on the fixed delivery rate during the peak water use period. During off-peak periods, such as early and late in the irrigation season, more water than necessary may be available. Excessive irrigation during these periods causes additional surface runoff and drainage problems.

The inflexibility of rotation delivery creates other disadvantages for the water user. These include:

- the set duration and frequency of delivery will be inappropriate for much of the service area, since varying soil conditions will cause different infiltration rates,
- crops must be restricted to those which are well suited to the fixed frequency of irrigation,
- disease and pests are more difficult to control,
- it is difficult to handle crops such as vegetables, which require prompt irrigation when fields are dry.

The major advantage of the rotation delivery concept is an easy main canal operation. Flow changes are infrequent and usually small in magnitude from day to day. Water level fluctuations are minimal. It is easy to maintain a balanced operation because the flow from canalside turnouts is constant and cannot be changed by the water users downstream. The main canal system can be operated by ditchriders (local manual control). Canal automation is usually not required because the flow remains relatively constant for long periods of time.

Considerable cost savings can be realized in the construction of canal systems when the rotation delivery concept is to be used. The size of the main canal can be optimized because the capacity of the canal prism is just large enough to convey the average flow during peak use periods. Since only a portion of the users will be receiving water at any one time, the canal capacity can be set at approximately 40 percent of the total water user allotment. Even greater cost savings occurs in the construction of the distribution systems. Rotating turnout deliveries among the water users can reduce the required maximum design capacity considerably.

Rotation delivery is a necessary concept when the main canal system has a limited or fixed quantity of water supply available at the canal headworks. All water users must share the limited water supply on an equitable basis. From an operational standpoint, the rotation delivery concept allows a main canal system to meet its objective of transporting water to the users with the least operating difficulty and expense.

2-10. Scheduled Delivery

The prevalent delivery concept in the United States is one in which water delivery is scheduled based on advanced notification. Individual water users turn in orders specifying the time of delivery and the quantity of water they wish to receive. These individual water orders are then compiled to predict the required canalside turnout flow changes All of the turnout flows are summed to create the total flow schedule for the canal.

Scheduled delivery causes some inconvenience, and possibly economic losses, to the water users. They must decide the quantity of water needed at their turnout in advance, one or more days prior to delivery, and then must adhere to that announced schedule. The schedule may interfere with other farm activities that develop at the last minute. If the anticipated water requirements in the plant root zone have been underestimated, the plants may become stressed or wilted. Overestimation of the quantity of water required can result in wasted water, runoff and drainage problems, and other agricultural inefficiencies.

Inaccurate estimation of water needs will result in decreased crop production. Usually, canal operators will make every possible effort to accommodate last-minute water schedule changes, particularly if there exists a potential for crop losses.

It is easier to operate a canal system when the deliveries are scheduled in advance. The main canal flow changes are predictable and normally can be accomplished without major water level fluctuations. Changes in the water depth are kept to a minimum by releasing flow into the canal before the actual time of the turnout deliveries. The response time is therefore accommodated in advance, diminishing the possibility of disrupting service to other canalside turnouts. An operation that minimizes water level fluctuations decreases the potential for canal lining and embankment failures.

The control of a canal system is also aided by the scheduled delivery concept. For local manually controlled canals, fewer ditchriders are required when the flow changes are predictable. Scheduled delivery can simplify the design of local automatic or supervisory control systems. By incorporating prediction, simpler control logic, control software, and performance requirements can be used. A simplified control system will reduce both acquisition and maintenance costs.

The maximum conveyance capacity of the canal with scheduled delivery will typically need to be only about 60 percent of the total individual water user demand. The individual water users' schedules can be adjusted to prevent them from exceeding the designed capacity of the main canal. Also, the watermaster can require water users to reschedule their deliveries if the cumulated water orders exceed the daily, monthly, or yearly contractual allotments.

2-11. Demand Delivery

The demand delivery concept allows users to take water from the canal system whenever they need it. The quantity of water delivered is only limited by the maximum delivery flow rate and total allotment restrictions. This concept of delivery is normal for municipal water distribution pipe systems, for example.

Demand delivery offers the maximum flexibility and convenience to the water user. Receiving water on demand also has economic value to the water user because there are no delays or quantity restrictions involved. The value of water supplied for irrigation can be much higher when it is provided to farmers exactly as needed rather than only as available. Maximum productivity can be achieved when water is applied at a rate to match field conditions and for the proper duration. This will permit earlier establishment of the crops, shorten the irrigation season, reduce weed and disease problems, allow more effective use of fertilizers, and result in larger crop yields. Precise application of irrigation water will decrease the total water use and reduce the need for subsurface drainage.

The provision of service to users on demand is both difficult and costly. Unrestricted delivery can create major operational problems for a main canal system, since it is difficult for an open channel system to respond quickly to unannounced flow changes. Unlike a pressurized pipeline, a long period of time is required to achieve a balanced operation following a flow change in a canal system. This slow response time can make it impossible for canal operations to keep up with demand changes, resulting in excessive depth fluctuations, wasted water, and disruption of service to other canalside turnouts. Also, a large water supply must be available at the head of the canal system to provide on-demand service.

A canal system which delivers water on demand will generally have high operation and maintenance costs. More sophisticated control will be required to provide the necessary response characteristics. Canal linings, embankments, and control system equipment will need more maintenance and repairs than a system using the other delivery concepts.

A canal system that will provide on-demand delivery is relatively expensive to construct. The canal prism needs to be sized to convey a larger peak demand, since there is no restriction on the number of users taking water simultaneously. The design capacity of the canal will typically be about 80 percent of the total entitled flow to all water users. This is based on the probability that a maximum of 80 percent of all the users will want water at any one time.

2-12. Comparison and Combination of Delivery Concepts

Each of the delivery concepts discussed above has advantages and disadvantages. The following chart summarizes these, and compares some of the major considerations relating to delivery alternatives:

Consideration	Rotation	Scheduled	Demand
User convenience	Poor	Good	Excellent
Irrigation flexibility	Poor	Moderate	Excellent
Crop yield	Low	Medium	High
Water use efficiency	Low	Medium	High
Ease of canal operation	Easy	Moderate	Difficult
Design capacity of canal as % of total water user allotment **	~40%	~60%	~80%
Complexity of the control system	Simple	Moderate	Complex
Cost of canal system	Low	Medium	High

** This percentage is approximate, since these values are highly dependent on water use patterns.

The demand, scheduled, and rotation delivery concepts are not mutually exclusive. Most canal systems will have the operational flexibility to combine concepts to some extent. Historically, water service on Bureau of Reclamation water projects is accomplished by a combination of scheduled and rotation delivery. The delivery flows requested by water users are distributed by the operating personnel as required. During peak demand season, on laterals which are pushed to maximum capacity, this may result in essentially rotation delivery. Combining the delivery concepts minimizes the construction, operation, and maintenance costs, and the objectives of the canal system can be achieved in an efficient manner.

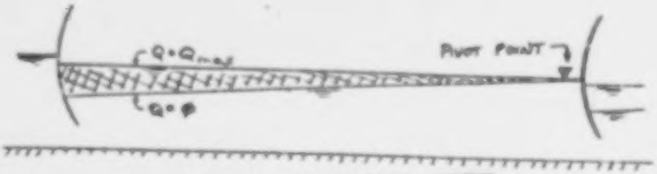
Demand delivery has not been commonly used as the primary concept of delivery on canal systems. However, on-demand service has been permitted in some cases when the flow changes were small and could be accommodated within the unused flow capacity of the canal prism. Modern control equipment can make demand delivery a feasible concept. The long term benefits to the water users justify increasing the capability of canal systems to accommodate demand delivery.

The delivery concept has a major impact on the overall operations of a canal system. It should be one of the primary considerations when evaluating alternative operations for either new or existing canals.

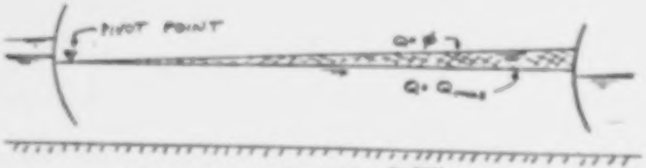
D. OPERATIONAL METHODS

2-13. Pool Operation Alternatives

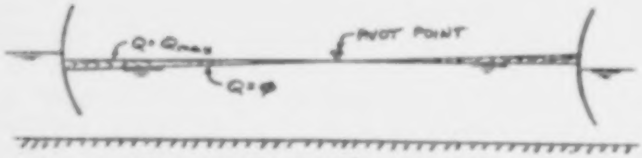
There are several methods which can be used to transport water downstream through a series of canal pools. Operational methods should not be confused with canal operation and control concepts, section 1-8. The method of operation determines how the water level varies in a canal pool to satisfy the operational concept. For this reason, it will sometimes be called **method of pool operation**. Operational methods are tied closely to the check gate operating techniques.


$Q = Q_{max}$
PIVOT POINT
$Q = 0$

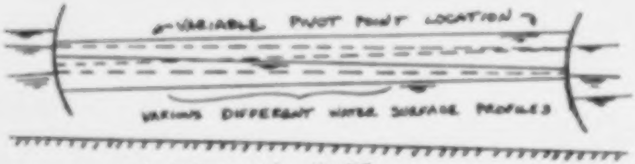
(a) CONSTANT DOWNSTREAM DEPTH


PIVOT POINT
$Q = 0$
$Q = Q_{max}$

(b) CONSTANT UPSTREAM DEPTH


$Q = Q_{max}$
PIVOT POINT
$Q = 0$

(c) CONSTANT VOLUME


VARIABLE PIVOT POINT LOCATION
VARIOUS DIFFERENT WATER SURFACE PROFILES

(d) CONTROLLED VOLUME

Figure 2-10. - Method of Operation

The method of operation is based on the location of the canal pool water surface pivot point:

- Constant downstream depth - The pivot point is located at the downstream end of the canal pool, figure 2-10(a).
- Constant upstream depth - The pivot point is located at the upstream end of the canal pool, figure 2-10(b).
- Constant volume - The pivot point is located near the midpoint of the canal pool, figure 2-10(c).
- Controlled volume - The pivot point can move within the canal pool, figure 2-10(d).

The basic method of operation should be identified for a canal system before control alternatives are evaluated. The location of the pivot point is particularly important when selecting a control method; i.e., local manual, local automatic, or supervisory control. Chapters 3 and 4 contain information on control methods and on the relationship between operational and control methods.

2-14. Constant Downstream Depth

Most canal systems use this method of operation, in which the water depth at the downstream end of each canal pool remains relatively constant. This method is associated with "conventional" operations and with local manual control, as discussed in sections 2.1 and 1-6(a). The primary reason why this method is so prevalent is that a canal can be sized to convey the maximum steady flow, and steady-state water depths should never exceed the normal depth at that design flow rate. The size of the canal prism and the lining freeboard can be kept to a minimum, reducing construction costs.

With a constant downstream depth, major turnouts are usually located near the downstream end of the canal pools. This allows the turnouts to be designed for a maximum and relatively constant depth in the canal. This helps to avoid problems in the delivery of water to the users due to low or fluctuating water depths. Wasteways, if any, will also be located at the downstream end of pools. Side-channel spillways and overflow weirs in the check structures can be set slightly above the maximum normal depth to prevent excessive depths.

When a constant depth is maintained at the downstream end of the canal pools, the water surface profile will essentially pivot about this point as the canal flow changes. (See figure 2-10(a).) This creates a storage wedge between different steady flow profiles. When the flow increases, the gradient of the water surface and the storage volume must also increase. Conversely, the storage volume must decrease for a reduction in the steady-state flow.

Because of these storage considerations, the constant downstream depth operational method is particularly effective when combined with the upstream operational concept (supply-oriented operation). There is a natural tendency for a flow change which originates at the upstream end of a pool to create the change in storage which is needed to keep the downstream pool depth constant.

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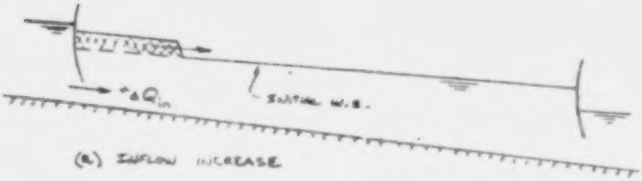
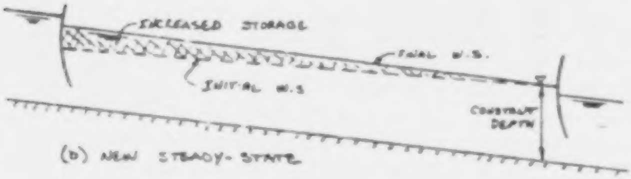
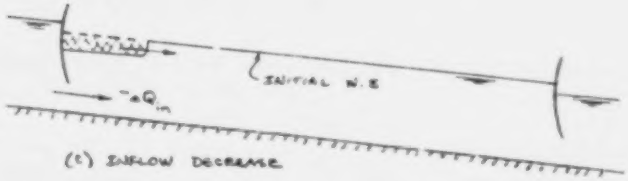
Figure 2-11 shows how a flow change at the upstream end of a pool causes the pool storage to change in a manner compatible with the constant downstream depth method of operation. When the pool inflow increases, figure 2-11(a), there is an additional supply of water into the pool. This additional volume of water supplies the increase in storage required to achieve the higher surface gradient, figure 2-11(b). When the inflow decreases, figure 2-11 (c), the pool outflow will temporarily exceed the pool inflow. This will cause a decrease in pool volume, as required to lower the water surface to the desired final steady-state flow profile, figure 2-11(d).


$+\Delta Q_{in}$
INITIAL W.S.

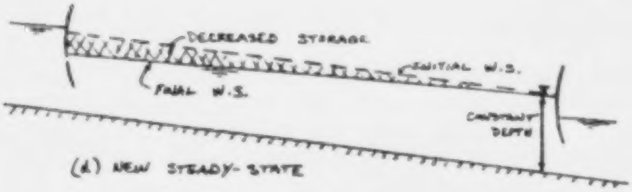
(a) INFLOW INCREASE


INCREASED STORAGE
FINAL W.S.
INITIAL W.S
CONSTANT DEPTH

(b) NEW STEADY-STATE


INITIAL W.S
$-\Delta Q_{in}$

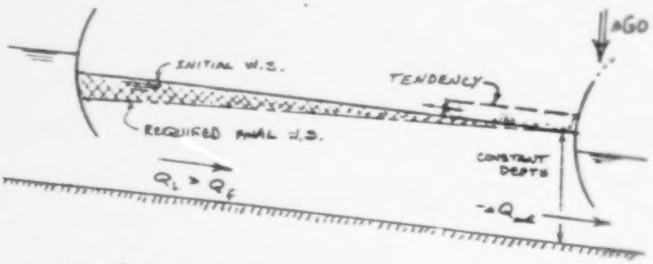
(c) INFLOW DECREASE


DECREASED STORAGE
INITIAL W.S.
FINAL W.S.
CONSTANT DEPTH

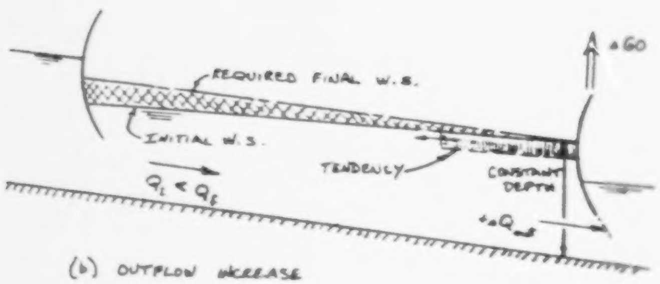
(d) NEW STEADY-STATE

Figure 2-11. - Upstream Flow Change with the Constant Downstream Depth Operational Method

The constant downstream depth method of operation has disadvantages when combined with the downstream operational concept (demand-oriented operation), because the pool storage must change oppositely to the natural tendency. As shown in figure 2-12, a flow change from the downstream end of a pool causes the depth to change in the wrong direction. When there is a decrease in pool outflow, figure 2-12(a), the tendency is for the pool storage to increase. In order to pivot the water surface about the downstream end, however, the pool volume must decrease. The same problem will exist with an outflow increase, figure 2-12(b), since the tendency to reduce the pool volume will be contrary to the requirement of increasing it.



(a) OUTFLOW DECREASE

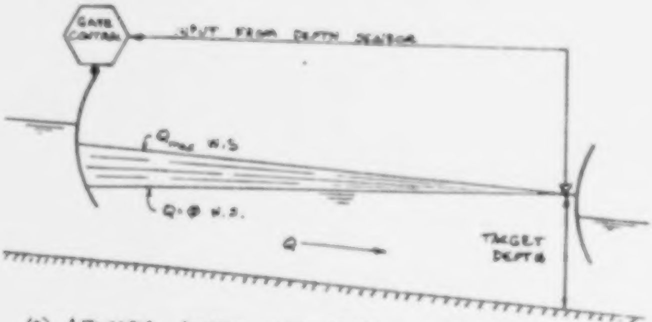


(b) OUTFLOW INCREASE

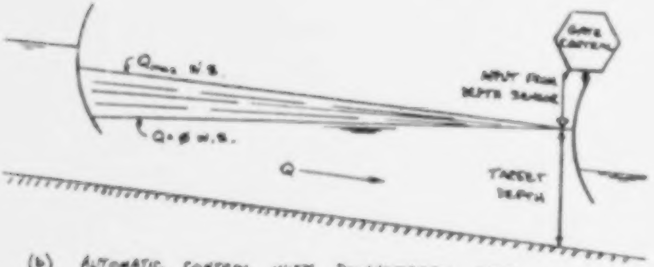
Figure 2-12. - Downstream Flow Change with the Constant Downstream Depth Operational Method

In a canal which operates with the constant downstream depth method, changes in demand are therefore much more difficult to handle than changes in supply. The canal will respond more easily and more quickly to supply-oriented operations than to demand-oriented operations. Flow changes which originate at the upstream end can be fairly large without creating problems. Flow changes which originate from downstream must be relatively small and gradual to avoid excessive depth fluctuations. Anticipation is often used to improve the system response to changes in demand, by creating upstream flow changes in advance of demand changes.

Any of the methods of control described in Chapter 1 can be used to implement the constant downstream depth operational method. Although local manual control has traditionally been used, automatic and supervisory control can also be implemented. The depth setpoint, or target, must be located at the water surface pivot point. The depth at this point can be controlled with either the upstream or downstream canal check gate, figure 2-13. The upstream gate would be controlled to satisfy a downstream operational concept, figure 2-13(a), and the downstream gate would be controlled to satisfy an upstream operational concept, figure 2-13(b).


GATE CONTROL
INPUT FROM DEPTH SENSOR
Q_{max} W.S.
$Q=0$ W.S.
Q
TARGET DEPTH

(a) AUTOMATIC CONTROL WITH UPSTREAM GATE TO SATISFY DOWNSTREAM OPERATIONAL CONCEPT


GATE CONTROL
INPUT FROM DEPTH SENSOR
Q_{max} W.S.
$Q=0$ W.S.
Q
TARGET DEPTH

(b) AUTOMATIC CONTROL WITH DOWNSTREAM GATE TO SATISFY UPSTREAM OPERATIONAL CONCEPT

Figure 2-13. - Control Alternatives for the Constant Downstream Depth Operational Method

2-15. Constant Upstream Depth

With this method of operation, a constant upstream depth is maintained by pivoting the water surface at the upstream end of the canal pool, figure 2-14. The constant upstream depth method is sometimes called **"level bank"** operation, since the canal banks must be horizontal to accommodate the zero-flow profile.

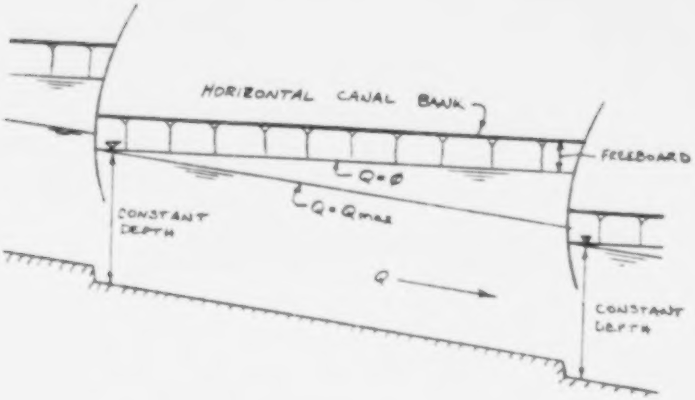
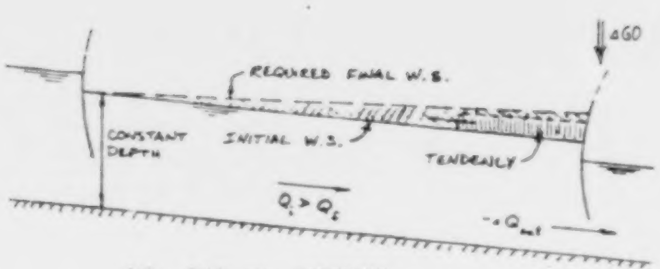


Figure 2-14. - "Level Bank" Operation

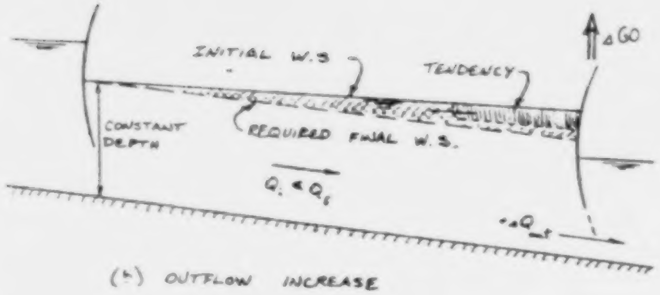
The construction of a level bank canal is the main drawback to this method. A level canal bank increases the cost of construction considerably, especially for concrete-lined canals. Most existing canals could not use level bank operation unless canal bank and lining were added to the downstream portion of each pool. Exceptions to this would be canals with very little elevation drop between checks, or those operating at flows well below the maximum flow capacity.

Turnouts can be located anywhere within a pool, since the canal water depth should always be at least the full flow normal depth. If a constant head is required, however, turnouts should be at the upstream end of the pools. Wasteways should be located similarly for best operation. If bypass weirs are to be included in the check structures, the weir crest must be higher than the zero-flow depth at the downstream end of the pool.

The constant upstream depth operational method is most effective when combined with the downstream operational concept (demand-oriented operation). Flow changes which originate at the downstream end of the pool cause the water depth in the canal to change in the direction needed to achieve new steady-state profiles.



(a) OUTFLOW DECREASE



(b) OUTFLOW INCREASE

Figure 2-15. - Downstream Flow Change with the Constant Upstream Depth Operational Method

Figure 2-15 illustrates the response of a level bank canal to changes in demand. A decrease in pool outflow will initially cause the downstream depth to rise, figure 2-15(a). This increase in pool storage will facilitate the change to a raised water surface profile, as required to pivot about the constant upstream depth. Conversely, an increase in outflow decreases the downstream depth towards the desired lower steady-state level, figure 2-15(b).

This excellent response to demand is the major advantage to level bank operation. Regulatory storage is essentially built in to the canal. At low flows, the additional water stored in the canal pools is readily available to supply increases in the demand. At higher flows there is storage space available above the water surface, where excess canal flow can be stored when there are decreases in the outflow. This allows the pool to react to demand, since the changes in storage within a pool can take place after downstream flow changes are made.

Level bank operation is inappropriate for supply-oriented canals. The operation would be inefficient and there would be no justification for the additional expense of the level bank construction.

Level bank operation can be accomplished successfully with any of the methods of control described in Chapter 1. Control can be based on maintaining the target depth at the upstream pivot point. When the downstream control concept is combined with level bank operation, each gate can be operated to maintain the depth immediately downstream of it. Figure 2-16 shows an example of local automatic control for a demand-based system.

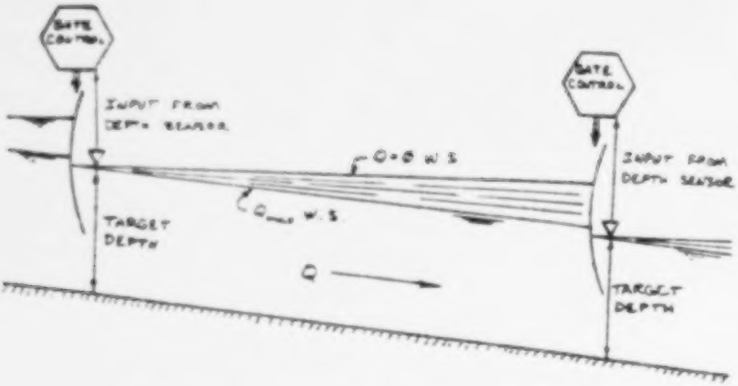

GATE CONTROL
INPUT FROM DEPTH SENSOR
Q=0 W.S
Q_{max} W.S.
TARGET DEPTH
GATE CONTROL
INPUT FROM DEPTH SENSOR
Q
TARGET DEPTH

Figure 2-16. - Local Downstream Control for the Constant Upstream Depth Operational Method

2-16. - Constant Volume

This operational method is based on maintaining a relatively constant volume of water in each canal pool at all times. The water surface will pivot about a point near mid-pool as the flow changes from one steady-state to another. The constant volume method of operation is sometimes called **"simultaneous operation,"** because the simultaneous gate operating technique is often used to keep the pool volume constant.

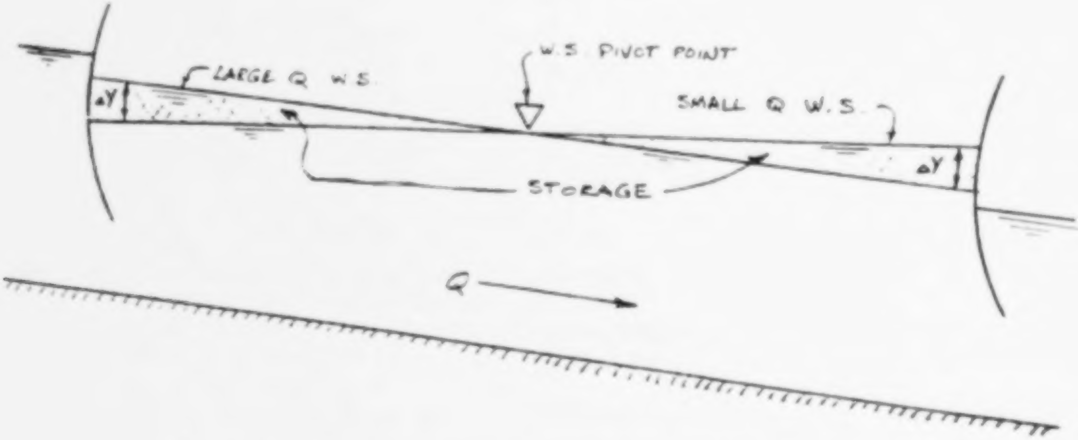

LARGE Q W.S.
W.S. PIVOT POINT
SMALL Q W.S.
ΔY
ΔY
STORAGE
Q

Figure 2-17. - Constant Volume Operational Method

As shown in figure 2-17, storage wedges will exist on either side of the mid-pool pivot point. For any given flow change, the volume change in each of these wedges is equal and opposite. When the flow decreases, the volume of water in the upstream wedge decreases and the volume increases in the downstream wedge. When the flow increases, the opposite occurs.

The main advantage of the constant volume method of operation is the ability to quickly change the flow conditions in the entire canal system. With the constant upstream depth and constant downstream depth methods, a great deal of time is required to either build up or deplete the storage in the entire canal system when changing the steady-state rate of flow. Constant volume operation avoids that lengthy delay, since the total volume of water in the canal system does not change significantly.

One disadvantage of constant volume operation is the additional canal bank and lining required at the downstream end of each pool, as compared to a conventional canal bank. This additional height, to accommodate the zero-flow water surface, is only about half of that required for level bank operation, however.

Another disadvantage involves the adjustment of check structures. It is very difficult to accomplish constant volume operation using local manual control. Local automatic control can be used, but the supervisory control method is best suited for constant volume operation. Using supervisory control, all of the control structures can be adjusted simultaneously from a central location.

2-17. Controlled Volume

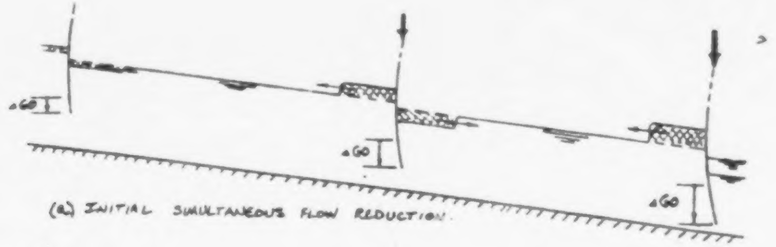
A canal system can be operated by managing the volume of water contained in one or more canal pools. The volume can be changed to satisfy operational criteria, allowing the pivot point to move within each pool. Since the operation is based on volume, either flow or depth may be used as the measured quantity. Unlike the previously discussed methods of operation, the water surface pivot point is of relatively little importance for controlled volume operation. In fact, the water surface may rise and fall without a pivot point, like a reservoir.

Controlled volume operation offers the most flexibility of any of the methods of operation. The canal operations can more easily adapt to normal, abnormal and emergency conditions, because there isn't a constant depth restriction. The operational flexibility is primarily restricted by the depth fluctuation limits.

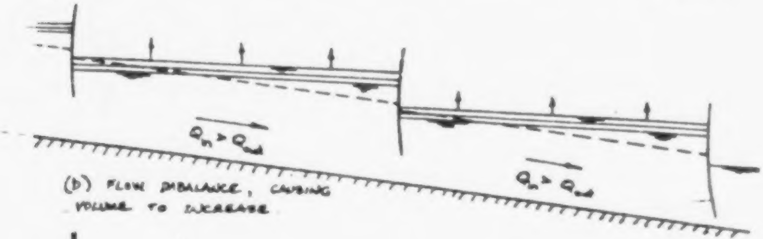
A canal system which operates with the controlled volume method has the capability to respond to a wide range of flow conditions. Sudden, large flow changes, which might otherwise require the use of off-channel storage or wasteways, can often be successfully controlled within the canal using the controlled volume method.

Controlled volume is a particularly suitable method of operating canal systems which include off-peak pumping considerations. (Electric power is often billed at a lower rate during "off peak" periods, when there is less power use, than during "on-peak" periods, when power use is high.) A substantial savings in power costs can be gained when pumping is maximized during the low-rate periods and minimized during high-rate periods. This can be accomplished by lowering the water levels in the canal pools during on-peak hours and raising water levels during off-peak hours.

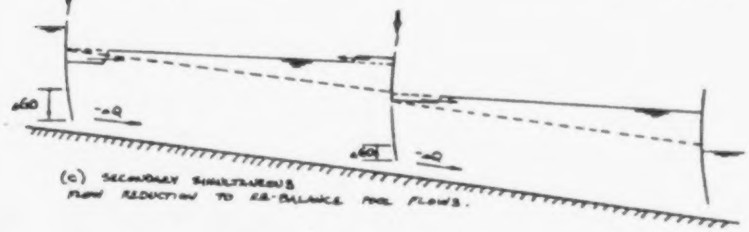
An example of controlled volume operation is illustrated in figure 2-18. In this example, there is a rapid outflow reduction at the downstream end of the canal. The check gates upstream are all simultaneously adjusted to reduce the canal flow, but the amount of flow reduction is smaller at each successive check. Therefore, the flow into each canal pool at the upstream gate is greater than the outflow, increasing the volume of water in each pool. After the volume has increased by a desired amount, additional gate movements can balance inflow and outflow to prevent excessive depths.



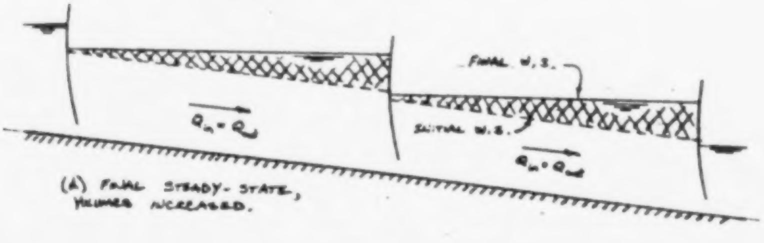
(a) INITIAL SIMULTANEOUS FLOW REDUCTION



(b) FLOW IMBALANCE, CAUSING VOLUME TO INCREASE



(c) SECONDARY SIMULTANEOUS FLOW REDUCTION TO RE-BALANCE POOL FLOWS.



(d) FINAL STEADY-STATE, VOLUMES INCREASED.

Figure 2-18. - Controlled Volume Operation Example

There could be several reasons for creating the operations in this example. The depth fluctuations can be managed to avoid rapid drawdown at the upstream end of the pools without wasting water or exceeding the maximum depth allowed at the downstream end. Limiting the drawdown can prevent lining damage which might have otherwise resulted from the flow change. Another reason might be to transform the rapid flow change at the downstream end into a gradual flow change in the upstream canal pools, by using pool storage as a buffer. If the rapid outflow reduction in the above example occurred during an off-peak power period, the storage increase would build up a reserve supply in the canal. The reserve could be depleted to meet deliveries at a later time when power is required elsewhere.

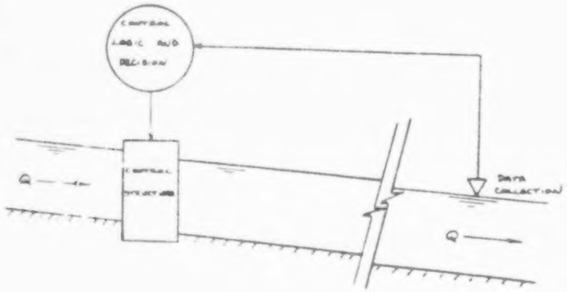
One of the disadvantages of controlled volume operation is that it requires the use of the supervisory control method. Without computer assistance, the complexities of controlled volume operations would require frequent intervention by operations personnel. Computer-directed control would use specially developed software to automatically control the check gates to maintain the desired volumes, requiring infrequent operator intervention. Another disadvantage of controlled volume operation is that it may require greater freeboard or a larger cross section for the canal prism.

CHAPTER 10. CONTROL FUNDAMENTALS

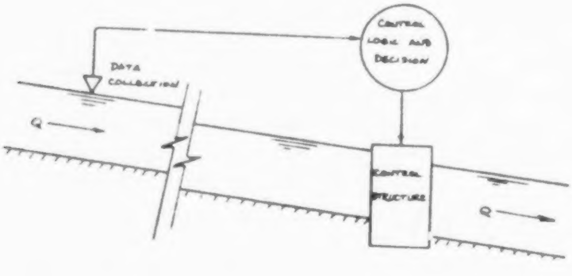
Most of the previous text has dealt with operations, which is the hydraulic behavior within a canal system. **"Control"** is the specific action of structures, intended to create the desired hydraulic reactions within the canal pools. Since the purpose of control is to enact the desired operations, it is logical to define the operations before designing a control system. When all of the pertinent operating criteria have been identified, it is appropriate then to consider the type of control scheme and equipment that would provide the best operation within the constraints placed on the water conveyance system.

10.1 Control Concepts

The control concept is defined by the location of the information needed for control relative to the control structure. This information can include the flow, depth, or volume at one or more points in the canal system, but it is commonly the water depth at a single point in the canal. The two basic control concepts reflect whether this information comes from downstream or upstream of the structure being controlled, as shown in figure 10-1.



(a) Downstream control concept - The control of structures is based on information from downstream.



(b) Upstream control concept - The control of structures is based on information from upstream.

The information on which control is based is usually measured or observed by operators or automatic sensors. In some cases, predicted or scheduled values are used. This information is then compared to a desired value (target) to determine if a change is needed. If so, logic is used to calculate the corrective action at one or more control structures. This control logic is often called an **"algorithm"** especially when it is defined mathematically. Canal operators have traditionally developed this logic through experience, but in recent years canal control algorithms have also been developed based on hydraulics and control theory.

Control concepts and operational concepts are very similar, but they are not identical. Operational concepts define the priorities of operation, yet control concepts deal with the actual enactment. Therefore, the relationship between operational and control concepts is a matter of compatibility, as shown in figure 10-2. Only combinations A and D in the figure are compatible.

		CONTROL CONCEPT	
		U/S	D/S
OPERATIONAL CONCEPT	U/S	A: Compatible	B: Impossible
	D/S	C: Inefficient	D: Compatible

Figure 10-2

To elaborate on combination B, it is impossible to control a supply oriented canal system (upstream operational concept) with the downstream control concept. When structures are controlled based only on downstream information, the system cannot react to changes in inflow. An example would be a canal with local automatic controllers which have the depth sensor downstream of each controller. Each check gate would only respond to downstream conditions regardless of what is happening on the upstream side of the check. If the supply to the canal is lost, due to a pump power failure for example, the upper end of the canal could dewater. Similarly, a rainstorm could increase the inflow to the canal through drain inlets, causing excessive depths. In these situations a shift to upstream control may be required.

A similar incompatibility exists between the downstream operational concept (demand oriented) and the upstream control concept, combination C. This combination is possible when excess flows can be wasted or stored if not delivered to the water users. Envision a canal with the demand delivery concept, so that the priority is to satisfy the user demands. An upstream control concept could be used to pass a maximum flow downstream, so that there is always enough water to meet the users' demands. Whenever the total demand is less than this maximum flow, the excess would have to be wasted or stored at one or more points along the canal. For most demand oriented canal systems, therefore, it would be inefficient to apply the upstream control concept.

It is possible to change the operation and control concept if the basic objective of the canal system changes. For example, a canal system can be designed to convey surplus flows during the winter months into a downstream storage reservoir. The objective during this operating period would be supply oriented. During the irrigation season, however, the canal system is primarily demand oriented. The operation and control concepts would need to change twice a year, which would require flexibility in the control system.

A continuous reach of canal cannot be operated and controlled with both the upstream and downstream concepts at the same time. A canal system may use the upstream concept in one reach and the downstream concept in another, however, if intermediate storage separates the two. Figure 10-3 shows an example of this.

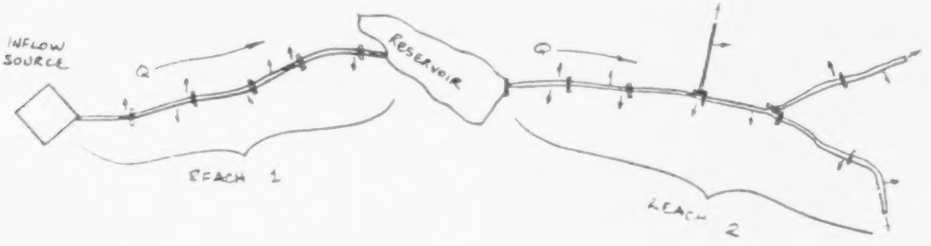


Figure 10-3

The first reach connects the main source to an intermediate storage reservoir, with deliveries along the way. Upstream operation and control concepts could be used here, passing all the undelivered water to the reservoir. The second reach could deliver water from the reservoir to the water users using downstream concepts.

10.2 Water System Control Methods

A canal system can be controlled by the following methods:

- Local manual - control is on-site by a human operator
- Local automatic - control is on-site by control equipment without human intervention

- Supervisory - control is from a central headquarters, with differing levels of participation by operations personnel

- Combined - control using a combination of the above methods

Each of these methods of control has advantages and disadvantages. Some of the characteristics of each method are described below.

Local manual control - This is the most prevalent control method, most commonly used in conjunction with conventional operations. The canal operators, or ditchriders, travel up and down the canal adjusting check gates and turnouts enroute. Labor saving machines, such as motorized gate hoists, are often used to assist the ditchrider, but the control is performed manually and separately at each control structure.

On a large canal system, many ditchriders may be required. The system would usually be divided up so that each ditchrider is responsible for a portion of the total system. An individual ditchrider would have to maintain the proper adjustment of all of the canal check gates and turnouts within his area. An experienced ditchrider can anticipate the arrival of flow changes at each of the structures which he controls, and time the adjustments accordingly.

The ditchriders can also monitor canal data, such as water levels and gate positions, reporting by mobile radio to the watermaster at a central headquarters. This allows coordination of operations among the team of ditchriders in the field. The quality of canal operations depends largely on the team skill.

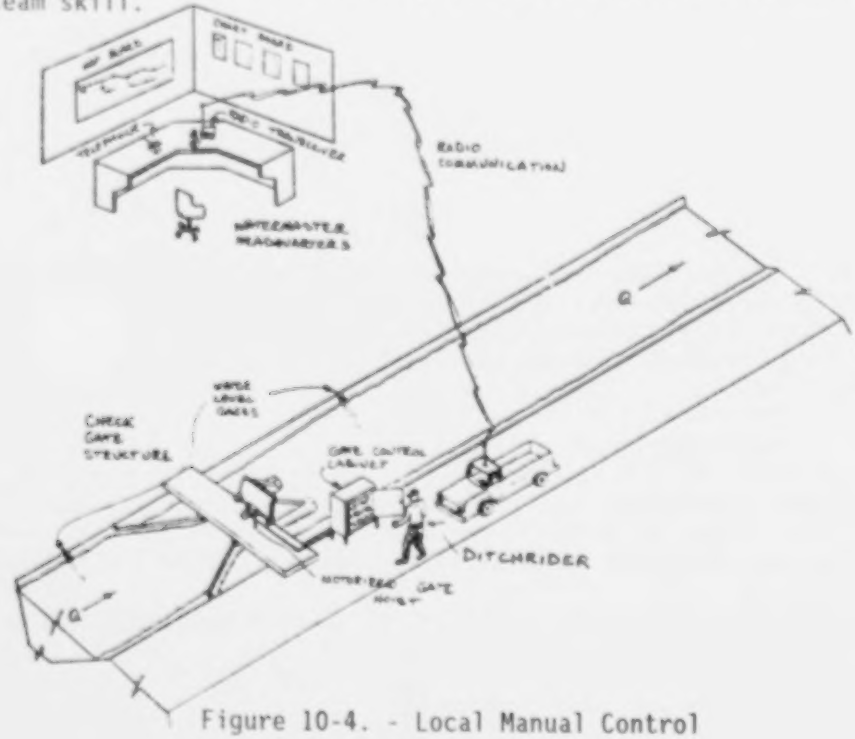

MAP BOARD
TELEPHONE
RADIO TRANSCEIVER
RADIO COMMUNICATION
WATERMASTER HEADQUARTERS
Q
WATER LEVEL GAGES
CHECK GATE STRUCTURE
GATE CONTROL CABINET
DITCHRIDER
MOTORIZED GATE HOIST
Q

Figure 10-4. - Local Manual Control

Local automatic control - This method of control is accomplished with automatic control equipment located on site, as depicted in figure 10-5. Various types of control equipment can be used, ranging from simple float-driven devices to complex arrangements of mechanical, electrical, and electronic components. Local automatic control is normally performed without operator intervention.

A local controller senses one or more pieces of information, such as water depth or gate position, and uses logic to create a control response. This control logic can sometimes be built into a mechanical device, such as a float and counterweight attached to a gate. More often, this logic will be expressed as a mathematical equation called a **control algorithm**. Control algorithms interpret the sensed information and calculate a desired control structure action, such as a gate movement. A local controller consists of equipment to execute the control algorithm.

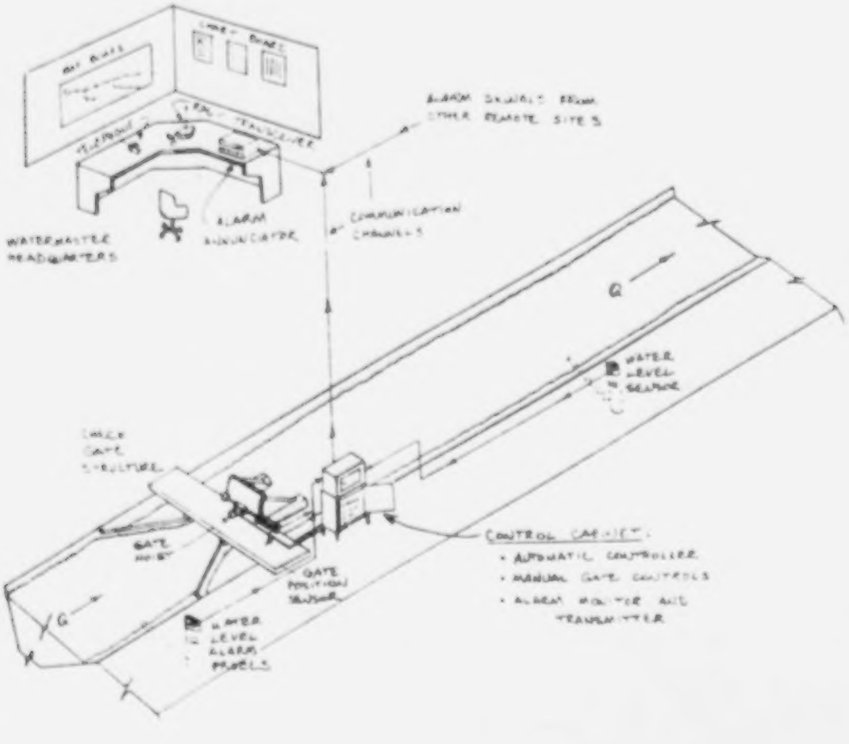

MAP BOARD
TREND BOARDS
TELEPHONE
RADIO - TRANSCEIVER
ALARM SIGNALS FROM OTHER REMOTE SITES
WATERMASTER HEADQUARTERS
ALARM ANNUNCIATOR
COMMUNICATION CHANNELS
Q
WATER LEVEL SENSOR
CHECK GATE STRUCTURE
GATE HOIST
GATE POSITION SENSOR
CONTROL CABINET:
• AUTOMATIC CONTROLLER
• MANUAL GATE CONTROLS
• ALARM MONITOR AND TRANSMITTER
Q
WATER LEVEL ALARM PROBES

Figure 10-5. - Local Automatic Control

Many different algorithms have been used in local canal controllers. Most of them are used to adjust a check gate based on the water level sensed at a single point in the canal, in an attempt to maintain a constant depth at that point.

The design of a local automatic control system requires both technical knowledge and practical experience. A good local controller will combine control theory with practical application to working canal systems. Most local control algorithms contain constants, or control parameters, which must be calibrated for each particular application. Therefore, a combination of control engineering and operator experience is needed to create and operate a system of local automatic controllers.

After the initial setup, local automatic control equipment should not require on-site adjustments by a ditchrider, although periodic maintenance and monitoring will be required. Most local automatic control systems should have a centralized alarm system to avoid disasters. An alarm system will notify the central headquarters of abnormal conditions such as control equipment failure, high or low water levels, local power outages, and communication channel failures. Communication is necessary between each check gate controller and the headquarters to provide the alarm information. A canal operator at the headquarters can monitor the alarm conditions and promptly initiate corrective actions.

Supervisory control - This method involves monitoring and controlling the canal structures from a central location, referred to as the headquarters or master station (figure 10-6).

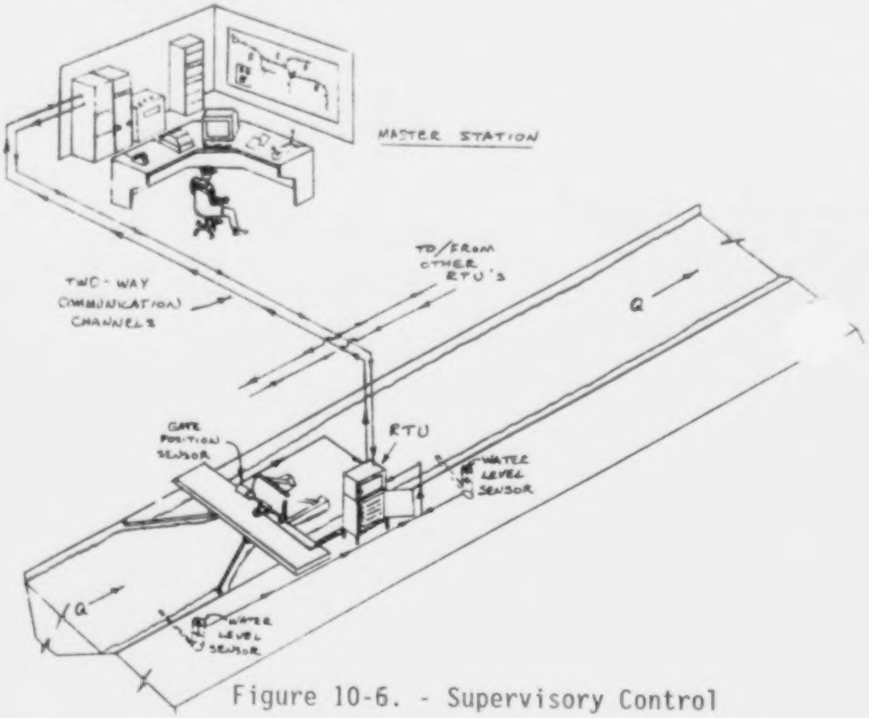


Figure 10-6. - Supervisory Control

Monitoring is the collection of data from various sites on the canal system and the presentation of this information for use in determining control actions. Data such as water levels, gate positions, flow, and pump status are collected at each remote location, including check gate structures and pumping plants. The information collected at all remote locations is transmitted to the master station, where it is analyzed and presented in a suitable format. Control decisions originate at the master station. Control commands are then transmitted back to the remote sites, creating control actions such as gate movements.

The supervisory control method requires equipment for data collection, communication, and control. Each remote site requires an RTU (Remote Terminal Unit). The RTU collects data, communicates with the master station, and controls the remote site based on information received from the master station. A communication system is required between each RTU and the master station, to allow two-way communication for monitoring and control. In addition to communication equipment, the master station will need equipment for monitoring, computing, and controlling. Due to the complexity of the hardware and software, and the requirement for a communication system, supervisory control systems are expensive to implement.

Supervisory control enables control decisions to be based on information from the entire canal system. A change in any portion of the system can be recognized promptly and the appropriate control action can be taken. This capability maximizes the operational flexibility of a canal system. Any of the methods of operation discussed in section 7 can be implemented with supervisory control.

The function of monitoring the canal system will include automatic features. Data will normally be collected automatically at the remote sites and transmitted to the master station. These data will then be disseminated, stored, and displayed at the master station. Computer-generated displays are often used to present this information in a graphical format. Operations personnel can observe present conditions and trends by calling up the desired displays. Automatic alarm capability is normally provided at the master station, to assure that operations personnel are made aware of extreme conditions.

The supervisory control method can involve varying levels of participation by master station personnel in making specific control actions and decisions. The actual control of the water system can be manual, automatic, or a combination of the two. Totally automatic supervisory control, without any intervention by operations personnel, does not exist. There is always a need for some human involvement in the control process. As explained below, supervisory control will be categorized as "automatic" when at least some of the control decisions and actions are accomplished automatically.

(1) Supervisory manual control - This method effectively transfers the ditchriders' duties from the field to a central location. An operator (watermaster) can monitor the data sent to the master station from each RTU, formulate control decisions, then transmit control commands back to the appropriate RTU's. These control commands are simple directives such as gate raise/lower or pump on/off. If there are two or more parts to a particular control sequence, the operator must manually execute each part.

Supervisory manual control is entirely dependent on the judgment of the watermaster. All control decisions and actions are accomplished manually, without assistance from automatically executed logic. The RTU's do not execute local automatic control algorithms, but simply carry out the supervisory commands received from the master station. This control method is appropriate when the operation of the canal system is relatively simple and when results of control actions are predictable. Frequent control actions are required by operating personnel, and operator attention typically will be required 24 hours a day.

(2) Supervisory automatic control - Although the amount of operator intervention may still vary widely, the control is of this type when the control process (not just monitoring) involves automatic features. There are several levels of supervisory automatic control, with the manual participation declining as the level of automation increases.

The first level can be called **setpoint** control. Instead of explicitly commanding a control structure's every move, the operator can establish a setpoint which is to be automatically achieved. For example, the operator can transmit a new gate position as a setpoint to the RTU at a particular check gate structure. The RTU will then automatically move the gate to the desired position without additional operator intervention.

Setpoint control allows a single operator command to execute a control sequence involving several steps. For instance, the command to start one pumping unit in a pumping plant may include unit selection, oil pressure and temperature checks, valve operation, and pump motor control.

Another example of setpoint control is the use of flow as the setpoint value. At a check gate structure, a flow-control algorithm will be quite complex, involving water levels, gate positions, and other properties of the check structure. At a pumping plant, flow setpoint control may include the automatic selection of units, speed control of variable-speed pumps, and valve operations.

In the setpoint control described above, an automatic process is initiated manually. The watermaster still makes the control decisions even though the implementation of those decisions is largely automatic. The second level of supervisory automatic control includes automatic **decisionmaking** logic, such as whether or not to make a control action, and what that action should be. New setpoints are automatically calculated, instead of being entered by operations personnel. Operator intervention is thereby reduced even further. At this level, operators will periodically adjust the automatic control and may switch to setpoint or manual control during emergencies.

Automatic decisionmaking control can be used to maintain a desired depth or volume of water. Control algorithms will involve hydraulics as well as control theory, similar to local automatic control algorithms. Operations personnel must provide a desired operating position, such as the target depth or volume to be maintained. The control algorithms may be executed either at the RTU's or at the master station. Regardless of the automatic controller location, the operator must be able to override the automatic control equipment and change the target values whenever he decides it is necessary. This second level of supervisory automatic control is typically used for normal operating conditions, making routine gate adjustments and freeing the operations personnel for other duties.

The third and final level of supervisory automatic control is called **computer-directed** control. This is the automatic control of the water system using specially developed computer programs (software) at the master station. These computer programs can use data from the entire canal system, instead of just from one location. Supervisory computer-directed control is applicable to the more complex water projects, and it may include optimization programs to allow a project to operate in the most efficient and economical manner.

Computer-directed control minimizes operator intervention. New operating positions can be established and transmitted to RTU's automatically. Consider, for example, a canal which uses the controlled volume method of operation. Master station software could automatically calculate new target volumes for all of the canal pools based on information such as water and power supply/demand schedules, system operational criteria, and past and present depths and flows. These target volumes could then provide the new operating position for the automatic decisionmaking control described above.

Combined control systems - Many canal systems use a combination of two or perhaps all three of the above control methods. The method of control can change either with time or with location. The most common combination is the use of manual control as a backup for automatic control, when power loss, equipment failure, or other emergency situations arise. Any automatically controlled system, no matter how sophisticated, will fall back to manual control as a last resort.

Manual and automatic control are sometimes used simultaneously at different locations on a canal system. Canals which are predominantly manually controlled may have local automatic controllers in particular locations. For example, a gravity turnout may require a local automatic controller to maintain a constant delivery rate if the main canal water level changes frequently. A ditchrider may not be able to visit the site often enough to make the necessary gate opening adjustments. When turnout flow changes are desired, the ditchrider would enter a new flow setpoint into the local controller instead of manually adjusting the turnout gate. Hence, this system combines the local manual and local automatic control methods.

Combining local automatic control with supervisory control will often have merit. By using RTU's with stand-alone control capability, the frequency of communication with the master station can be greatly reduced. This can save master station computers and/or operators from constantly sending control information to each check structure on the canal. With this combination, supervisory control may be used primarily to adjust the setpoints or the control parameters in the local control algorithms. Supervisory control can also override the local control whenever abnormal conditions arise. The system could operate similarly to an automobile cruise control, wherein the local controllers cruise except when overridden by the master station. This combination provides an additional benefit of allowing normal control to proceed without interruption during a temporary communication failure between RTU and master station.

Session 2-1

Scott Coulson

Introduction

The use of computers today, for water management or accounting, significantly improves management's ability to monitor an entire system from orders, to billing, to the delivery of water to customers. Automatic and remote controls for water delivery and storage systems improve service, reduce water waste and operational costs, increase flexibility, allow timely responses to emergencies, make system upgrades easier, and provide an overview of the entire system for operational considerations of the past, present, and future. The computer used for billing, accounting, and payroll will not be the same computer used for the Supervisory Control and Data Acquisition (SCADA) system, but will be as valuable to its users. The ability to retrieve records by a few strokes of the keys saves hours by not having to look through file cabinets. Computers and electronics in general are here to stay.

There have been many studies and opinions given on the subject of automatic and remote controls of equipment used in water management and recordkeeping or accounting. Ten years ago there was even some doubt about automatic controls. Today if there is any doubt it has to do with how much control or how much memory is needed for storing information. But everyone agrees that some automation is necessary. Some applications will have a longer payback time than others, but it will pay for itself. There have been disputes and challenges as to how electronic systems should control different functions; but one thing has remained constant, once the electronic systems have been installed they remain. They will be improved, repaired, adjusted, and sometimes cursed, but they will not be removed.

The electronic equipment that is discussed in the notes for this session are primarily directed toward irrigation functions, but they are applicable to other areas as well. The Coachella Valley Water District (CVWD) uses the same electronics in its applications in domestic water, sanitation, water recharge, and flood channel monitoring. The equipment selected for use in the various areas must be versatile. This reduces costs in training, repairs, and spare parts. The computer equipment that is used in accounting is also used in engineering, billing, customer service, payroll, and by various personnel in the district to keep records, to transfer records, and for electronic mail. The selection of the proper equipment for your system is an important one. For districts that are just beginning, it might seem inconsequential which electronic equipment is selected. For the already initiated, you can look back and see the significance of the decision. This decision will frame the future of your district. The equipment that you buy today will be the equipment that you build on tomorrow. Most electronic equipment can be used for many years. This is one of the ways it pays for itself. CVWD had equipment that lasted over 15 years and was still functional at the time it was replaced. So consider your choices carefully; this could be the beginning of something really big.

This portion of the notes for Session 2-1 contains an overview of how to monitor levels, flows, and gate and valve positions. It will also cover communications, equipment protection, how to manage water with electronics, how to select the equipment through specifications, and how to maintain your own equipment. In addition, accounting, billing, customer service, payroll, and engineering functions will be discussed. There are many other aspects of electronics, and hopefully as you begin to learn more about them you will want more information.

LEVELS

The information that should be available at your headquarter's office is the level of a canal, reservoir, or lake. (These are the most common.) To obtain this information, the level figure needs to be converted to electrical data. This is accomplished by a "transducer." This, by definition, is a device that will change one form of power to another. In plain words, it converts a hydraulic level to a current or voltage that is fed into an electronic device. One such device is a simple float and weight where a chain is placed over a wheel and between the weight and float, and then connected to a potentiometer (variable resistor). As the float moves up and down with the water level, the potentiometer moves. With voltage applied to this potentiometer the change in voltage or current can be monitored to sense the level change in the water. This is a simple mechanical transducer which has been used for many years in various areas and is still used in some applications.

As electronic technology developed, an electronic transducer came to be one of the more commonly used devices. It connects to a pipe or is placed in a reservoir. The internal diaphragm into which strain gauge resistors are diffused is then interconnected to form a pressure sensitive resistive bridge. This is pretty basic; the water pressure against the diaphragm makes a very slight movement and affects the balance in the bridge and changes current flow. This change is sensed with some other electronic equipment that will convert it to a form that is suitable for forwarding to headquarters. This can also monitor air pressure. So if a transducer must be mounted above the water level, a pipe can be placed to the bottom and a small compressor connected to the pipe and transducer. The transducer will monitor the pressure required to produce an air bubble from the pipe. The higher the water the more pressure required to produce the bubble. This will be monitored and converted to current flow. This in turn will be converted for transmission to headquarters.

Another method of level monitoring is the sonic sensor. This device transmits a signal and monitors the returned signal that is "bounced" off the water level. It is positioned directly over the water and works best on smooth water surfaces. This unit usually produces a current flow that can be converted and then transmitted back to headquarters.

Other methods are capacitance strips or probes. These are submerged in the water and the capacitance is changed by the water surrounding the probe or strip. They also produce a current flow.

To transmit this information to headquarters, it must be converted to a form that is compatible with long distance transmissions. Current is only good for distances less than a mile. Anything over that must be converted. One way is to convert it to a digital word which makes it easy for a computer to read. An analog to digital converter should be used. This converts currents to "ones" and "zeros" which are digital words. They can then be transmitted, via a modem, by land line or radio to headquarters. The computer at headquarters will convert it back to a number that can be read on the Cathode Ray Tube (CRT) screen.

DIGITAL WORD:
ONE------
ZERO-----

Another method used is an RC time constant. This is usually used with a "pulsed duration" system. Depending on the current flow, the Resistor/Capacitor circuit will charge or discharge for a period of time. This time period is proportional to the water level. For this time period, a relay will be energized for a like period of time. This is used to turn on an audible tone frequency which can be transmitted on land line or radio. At headquarters a receiver will accept the tone and energize a relay for the same period of time. That in turn will feed an RC circuit that will charge up during the "on" time. This will be sensed by the electronic circuit to produce a voltage output proportional to the water level. This can be connected to a meter, chart recorder, or computer interface. Each of these communication methods and sensors can be used in many different applications. Each has a speciality that it is best suited for, but they are versatile enough that they can be used in more than one application.

POSITION INDICATORS

The most commonly used position indicator is the potentiometer. This is a variable resistor that has a shaft attached to the wiper of the "pot." As the shaft moves, the resistance of the pot will change, varying the current or voltage. This is then converted, just like the level, and reported to headquarters via one of the communication methods. The shafts of the potentiometer are commonly connected to the shaft of a gate or motor shaft of a valve. As the gate or motor turns, the shaft is moved changing the current flow. The most difficult problem with position indicators is how to connect the pot shaft to the gate or motor shaft. Shaft couplings that reduce in size are the most common. But if the shaft is exceptionally large, a smaller bolt can be welded to the gate or motor shaft and the coupling attached to the bolt.

Another method is to connect a cable to the bottom of the gate along with a pot with a long shaft. The cable should be placed around the pot shaft and connected to a counterweight. As the gate opens, the counterweight will pull the cable and move the shaft, and in reverse, when the gate closes, it will pull the cable and turn the shaft in the opposite direction.

One philosophy to follow is: Keep it simple. More sophisticated methods can be employed on complicated operations, but they are not necessary in most cases. Once you have a basic understanding of electronic monitoring, and as your system becomes more complicated, you will be able to understand the functional operation of more sophisticated electronic equipment.

COMMUNICATION

Once the equipment is in the field and at the headquarter's office, communication becomes the key issue that you must face. For very large systems, the best methods are a combination of satellite, microwave, UHF, and/or VHF radios and land lines. It is recommended that they be owned, operated, and maintained by your own company (with the exception of the satellite), if the project is large enough to warrant this type of operation. If the system is smaller, then microwave or UHF or VHF or a combination of these radios would be adequate. There is one other option that should be avoided, but sometimes there is just no other way. The telephone company. At times it can be of service, but the problem of trying to track down a problem or getting the company to respond to a problem can be frustrating and delaying. There are companies who install their own cable, but the drawbacks to this can be many man-hours spent repairing the cable, especially if it is in an area where the cable is constantly being dug up due to construction activities. Of these options the CVWD has selected microwave, UHF, VHF, telephone line, and our own cables. They all have their advantages and disadvantages. You must pick the best one for your application and budget.

As each option is considered, you will see that cost is a contributing factor in selecting a communication method. Satellite is the most expensive, but in applications where there are several states to cover, it can be cheaper than installing a microwave system or using telephone carriers. The satellite will easily cover large sections of the United States.

If the area of application is smaller, say 1,000 square miles, then a microwave should be considered. If there is a lot of information from many sites, maybe a combination of UHF, VHF, and microwave would be applicable. This could be a series of microwave stations with radio, telephone cable, or privately own cable branching out from the microwave sites. This will allow many sites to report to one station and then get incorporated into the larger microwave system.

If the system is located in a city or rural area, UHF, VHF, and/or telephone cable might be in order. UHF and VHF are pretty reliable in areas where there are no obstructions; therefore, they can be used in the city for short distances, or in the country where the terrain is open. If there are many sites, then it requires some sort of control to send and receive information from one central site to remote areas. It would be advantageous to use the same frequency at all sites and control the action of the sites by using a process controller or computer to manage the action of sending commands and receiving reports. The Federal Communications Commission (FCC) has several pairs of frequencies in the 900 MHZ band that are very suitable for this application. The Utilities Telecommunication Council (UTC) is encouraging companies to use these frequencies.

Again, there are several options available for combining the different types of communication methods to meet your needs. Generally speaking, the more area to be covered the more complicated the communication system. But it should be kept as simple as possible. Interaction with other companies should be minimized as much as possible. The more you must rely on others the more disappointed you will be with your system's reliability.

EQUIPMENT PROTECTION

One of the most damaging elements to affect electronic equipment is MOTHER NATURE. She can do more damage quicker than any other single source imaginable. Lightning is the most powerful and quickest to strike. Transducers, radios, microwaves, cables, and computers are all susceptible to damage from lightning. Fortunately, there is protection from this damaging element. The common name: lightning protection devices. These devices vary in technique.

Basically they react very, very fast, and short the high voltage to ground and then automatically reset for normal service. The devices will differ for equipment that is producing 4-20 milliamps and radio equipment. They are different for microwave and cable, but the point is, it is available and they do work. Equipment should be protected as much as possible. Microwave may cost $20,000 and a lightning protection unit only $60. A lot of lightning protection equipment can be replaced over the life span of the microwave, and without this equipment the microwave might have to be replaced.

Another source of damage is from power surges. Surges caused by MOTHER NATURE or from the power company can cause damage to electronic equipment. Again, there are surge protection devices that do a very good job keeping the peaks and valleys out of incoming power. In this same area, it is essential that the computer be fully protected from lightning and power surges. It is also essential that the computer does not just turn off due to lack of power. There is no way to predict what the computer might be doing and information could be lost or worse the computer could sustain serious damage by the immediate loss of power. There is a recommended method of protection called Uninterruptable Power Supply (UPS). Basically, this is a unit with batteries that is constantly charging and supplying voltage to an inverter. This inverter will change the D.C. voltage from the batteries to supply A.C. voltage to the computer. If power is interrupted, the batteries will continue to supply voltage to the inverter for a period of time necessary to allow an orderly shut down of the system or allow time for the backup generator to start up.

All of these units and devices are essential for a reliable system. The more a company becomes involved with electronics the more dependent it will become on the system. This will increase the priority of keeping the system on-line at all times. This means that it must be protected; so start now.

WATER MANAGEMENT

With the ability to see the whole system, managers can better supervise the use and handling of water in the system. Even if it is an isolated system or a small one, it will still allow the management of the water in that system to improve significantly. Just the ability to know what is happening without having to travel long distances will be helpful. Most managers can make good decisions with all the facts in front of them. Electronics can give you those facts. In addition to on-the-spot decisions, information is available from the past which can be used for future considerations. By making comparisons of past and present operations, future needs can be determined and projects planned. Another area where this becomes important is safety. When misfortune falls upon your system, you will be able to determine the reason why. This will allow the scenario to be analyzed and precautions can be taken to prevent a reoccurrence of the same or similar problems.

The CVWD canal is divided into ponds. The majority of the ponds deliver water to the agricultural community. Electronics are used to insure water deliveries to each pond at a particular time. At 5 p.m. on any given day, the water orders are closed for the next day. Based on meter numbers, the orders are associated to a pond in the canal system. The information is fed into the computer and, at midnight, the computer starts to make changes to gates that will provide water to the furthermost pond at 6 a.m. As the hours from midnight progress, so do the changes for the delivery system. The gates are automatically opened to allow water delivery changes to pass through the system. If the water orders are reduced, then the reverse happens. The gates will be closed to reduce the water at a particular pond by 6 a.m. This reduces water losses, improves water service to the customer, and insures that the changes are made in a timely manner without human interface. If the control operator has other duties that require his or her attention, the changes might not be made in a timely manner; the computer never forgets. As more water delivery points are added to the system, additional management controls will also have to be included. The more control the more efficient the operation.

Another advantage to overall system management is that it allows effective responses to emergency conditions. When situations occur that require several changes in different locations, the system allows one person to make all of the changes without leaving his or her chair. In the late summer of 1988, there was a flash flood that plugged the CVWD canal. The control operator noted that there was a problem, dispatched crews, and made gate changes on over 100 miles of canal. The result was no water lost, no damage to residences or businesses, and only a minor inconvenience to customers for 18 hours. On the other hand, if a person had to go see why there was no water and had to drive over 100 miles to close gates, there is an excellent chance that the whole canal could have washed away. Customers could have been out of water for days or weeks, and there could have been millions of dollars of damage done to businesses and residences in the area. This is an excellent example of how electronics can benefit a district in managing its water and system.

AUTOMATION IN ADMINISTRATION

The accounting of money and water is very high on any manager's list of important things to do. Computers can make this job much easier and will make access to records much quicker.

The most important subject to any employee or manager is payroll. The computer can keep track of regular and overtime hours worked. It will automatically figure deductions; it will keep long term records; print checks; make reports on sick leave use, vacation, overtime; and the list goes on. The system is versatile and can help keep track of any item that might be important, including interest from bank accounts.

All of the reports can be used by management to spot trouble areas. You can easily spot the employee that takes Mondays and Fridays off sick. Projects are just as easy to track. The system is very flexible. Procurements, deliveries, payments, billing, and work progress can be kept on the system. This makes information available to all who need it and the status can be quickly checked at any time. If policy allows, you can even call in from a remote computer site to check project status without having to track down the right person. It makes information available.

Precautions must be taken not to allow just anyone into your system. This can be done by a security setup that requires passwords or call back technique. These two are the most common ways of securing your system. The first is a secret password. This can be changed from time to time and it will keep most people out. But the sharp "hacker" will get by the password check. The call back method is more secure. The computer is programmed to only call certain numbers. You then call in with your particular password and then hang-up. The computer will look up your number and call you back. In this manner, even the "hacker" that gets by the password will be cut off and some other telephone number will be called. This method has proven to be very reliable.

Once you have either local or remote access you can still take further precautions. Only certain files can be viewed; the changes can only be made by key personnel. Other files can be restricted. For example, personnel records or general manager's files would normally be secured. One or more of these files may be of interest to another user and the authority to view one of the secured files can be allowed by the system manager or the originator of that particular file. This allows the system to be fixed but flexible.

The uses for administration are not limited to accounting, billing, and the other areas already covered. It can be used for many more features, all of which you will become familiar with as your knowledge of data processing increases. Magazines from computer manufacturers and software magazines are very helpful. The capabilities of a computer are mostly limited by the user's imagination. If you can think of it, a computer can do it. And the computer can do exactly the same each time.

One more item, the SCADA computer should not be considered a data processing computer. If both SCADA and data processing are required, then you need two systems. The two can be tied together, but the data processing system should only be receiving information from the SCADA system. The reason for this is that data processing takes time. If users are tied to the SCADA system, it can slow the system down to a point where valuable information from the sites in the field would be missed. The SCADA system should only have one user on it - the control operator. With a data processing system, as the numbers of users increase, the response time to any one user is longer because the system has to be shared. The same happens if there are several users on the SCADA system. The SCADA system should send its gathered information to the data processing system. Then it can be manipulated, reported, sorted, and viewed by as many users as the system can manage. This will keep the SCADA system gathering information all the time and nothing will be missed.

SPECIFICATIONS

A specification must be written so that the item desired is what is received. It is the communication tool that is the determining factor in disputes or disagreements. There must be no room for interpretations; it must be accurate and each detail must be explained to the contractor in writing. Nothing that is communicated orally is acceptable. If there are any changes to be made, they should be in writing.

As for the specification, it should begin with an overview of what the project is to accomplish. This should be in general terms to give the reader an indication of what is expected. Then, in more detail, the specification should stipulate the exact actions required from an operational, functional viewpoint. Then the contractor responds with his idea of what equipment will do the job based on the information provided. With different contractors, a variety of types of equipment will be available for consideration so the best buy for the money can be made. The other alternative is to specify the exact equipment. The drawback to this type of specification is that it may limit some potential vendor options. In later projects, you may have to make specific purchases so that what is needed is compatible with what is existing, but on the initial project, let the contractors do all the researching. They are the closest to the ever changing electronic market and they will be the ones to help set up an appropriate telemetry system.

Just don't listen to the sales people; stick to the facts. In the specification, require bidders to submit "cut sheets" that describe the equipment and detail specifications for the equipment that is recommended. If there is a discrepancy between what the sales person says and the "cut sheet," believe the "cut sheet." Sales people have a tendency to orally exaggerate what equipment can do; "cut sheets" do not.

Once you have the specification out and the bids returned, a team can be formed to evaluate the equipment and the styles of control and monitoring. This team of evaluators should include people from different points of view. For example, a manager, an operator, a technician, an electrician, and someone from the operations area who understands how the manual system operates. These people will need time to read and compare all of the bids. Some of the bids can be eliminated by cost, but there should be several that will be within your price range. It is also possible to have a two-stage bidding sequence. In this case, the number of bidders is narrowed to two or three and then they are asked to rebid. At this point in the process, any mistakes made by the purchaser or the bidders can be corrected. If there was something left out or miscommunicated, it can be corrected at this time and, of course, in writing. The specification is a tool just like any other. Use it to your best advantage. One last thing, the more reliable contractors will work with you, but you must be careful when selecting a contractor. Get references and check them. Many contractors use the same equipment at different jobs. When a potential contractor specifies a certain piece of equipment, ask to visit a site where similar equipment is installed. This will help to insure that the contractor is reputable and the equipment dependable. When visiting a site ask about the procurement process and if they would do anything differently the next time. There is no rule that says we have to make the same mistakes as everyone else. Also ask about the support services of the vendors and manufacturers. Support may not seem important now, but after a few years when the equipment starts to have problems, you may find this information very important. Documentation is important, also. Without schematics, it is almost impossible for a technician to repair electronic equipment. The same applies to program listings; if it is necessary for a programmer to change or troubleshoot a program, he or she will need this information. Some companies are reluctant to supply these items. The purchaser should insure that this information is required by the specification. And be careful. Some companies supply block diagrams instead of schematics. If you do not know the difference, find someone who does. This is really important for your district. Be sure to specify schematics and program listings.

SELF MAINTENANCE

Probably the most important aspect of the project is the personnel. The need for hard working, knowledgeable people is a key factor in how well a system is operated and maintained. For technicians, it is a good idea to require they be certified. There are several certifications that are available and most of them are good. The most common are National Association of Business and Educational Radio Service (NABERS) or National Association of Radio and Television Engineers (NARTE). There are also some good technicians that are licensed by the FCC. For system operators, you will need people who are familiar with every aspect of the system before it is automated and they must be trainable in the operation of the system after automation. Personnel with electrical experience are also important. This may include pump maintenance personnel. This group will need training and will need to take on some additional responsibility. Since the automation must interact with their equipment, they must understand how it will affect their portion of the operation. Certifications are a good indicator of a person's ability to learn; a test should be required before hiring. If you are able to find a few key people in your organization, you can begin to build a group that will enhance your system's dependability. One other alternative to consider is training. There are companies that will train technicians and operators, and there are companies that will maintain your equipment; both are expensive. The best scenario is to have your own personnel train and develop with the system. If the district facility is remote, this becomes even more important. If the site is critical, then it is imperative that in-house personnel be able to maintain it. If there is a problem, in-house personnel will need to respond immediately. If you have a maintenance contract, the response time will be considerably longer and if there is a storm that caused the problem, you may have to wait in line with other customers. The more you must rely on others the more disappointed you will be with your system's reliability.

CONCLUSION

With all of this in mind, the design, the safety, the operation, and the dependability, you must proceed toward your goals. These goals can be an exciting and interesting part of your job OR they can make your life miserable. The fundamental steps laid out in this document have been very successful for the CVWD. This information has been shared with many organizations and CVWD has supported them in their successful efforts to establish or improve their systems. The same assistance is available to your district. Please contact the CVWD if you have any questions about any portion of your project.

We will be glad to help in any way we can. Call 619-398-2651.

GOOD LUCK!

Water Systems Operation and Maintenance Workshop - 1990

WATER SYSTEMS MANAGEMENT, PART 2

Session 2-2

Stan Conway and David King

Water Management and Conservation (WMC) Program

The Bureau of Reclamation, as the principal water agency in the Western United States, is responsible for the development and conservation of the Nation's water resources. The Bureau programs, including the WMC program, most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water user organizations, and other concerned groups. The Bureau's authority to conduct the WMC program is found in the general provisions of Reclamation Law which authorized the Secretary of the Interior to plan, construct, and operate works for the storage, diversion, and development of water for the reclamation of arid and semiarid lands in the 17 contiguous Western States. The general provisions encompass the WMC program and other measures on the irrigation projects developed under Reclamation Law. The WMC program is necessarily concerned with the privately owned lands served by Reclamation projects under the jurisdiction of the Department and the Bureau.

Water Management and Conservation

Introduction

A major O&M goal is better management and more efficient use of water, energy, and other resources on our operating irrigation projects. The WMC program was developed as a nonstructural means to promote improvements in project and onfarm water systems and management practices. In addition to reductions in water use and losses, the program objectives include energy conservation and the protection and enhancement of water quality. As provided in the draft Reclamation Instructions, the program covers three general types of activities: (A) developing or adapting new technologies for efficient use of water, efficient use of energy, and protection of water quality in irrigation operations, (B) promoting the use of improved technology and management practices by water users, and (C) technical services and assistance to water users organizations and to allied Reclamation programs (e.g., water quality, project planning, etc.). The instructions place considerable emphasis on water measurement and accounting, evaluation of irrigation efficiencies, and evaluation of the program itself.

Over the past 15 years, the Bureau of Reclamation through its WMC program, has developed some practical tools designed to enhance irrigation district operations. Two of the tools dealing with scheduling irrigation water are:

1. Onfarm field irrigation scheduling provides timely information on the water needs of the crop. This information includes forecast irrigation dates and amounts of water required to refill the root zone on the forecasted dates on a field-by-field basis.

2. Distribution system scheduling schedules the movement of water through a distribution system to meet crop water needs in a timely and efficient manner.

An irrigation district utilizing either or both of these tools may realize the following benefits:

a. Higher crop yield
b. Better crop quality
c. Lower production costs
d. Lower district operation and maintenance costs
e. Improved criteria for planning and development of future needs
f. More favorable environmental impact

Distribution system scheduling involves the compilation and organization of water deliveries to the individual fields throughout the project in a timely manner to provide the proper amount of water required to meet crop water needs. This is referred to as system scheduling. Nearly all districts or projects are now operating under some form of system scheduling. The ditch-rider regulating water on a lateral, simple as it may be, is a form of system scheduling. The Bureau of Reclamation is striving to improve system scheduling by incorporating the actual farm water requirements into this schedule by providing more precise information on crop water needs with the field irrigation program. This may be in conjunction with systems improvement and necessary modernization of facilities, structures, and operating policies.

For those operating a district or project, the most important phase of this program is the incorporation of onfarm water management into the distribution system management to achieve total water management at the project level. To achieve improved onfarm management, substantial changes in present irrigation and water delivery practices could be expected; the irrigator will need to have better information as to when he should irrigate a particular crop and how much water he should apply when he does irrigate. When available, a field irrigation scheduling program provides this information. Along with this information, he often needs technical assistance in preparing his land for irrigation and in distributing the proper amount of water uniformly over the field.

Scheduling of water to fields for optimum crop yields must take into consideration the constraints and limitations imposed by the farm delivery system as well as those of the project distribution system. In some cases, field irrigation schedules may need adjusting so that farm delivery requirements do not exceed the capacity of the farm's delivery system. Whenever the recommended date of the next irrigation of a field is changed, the amount of water to be applied is also recalculated in order to maintain the best irrigation efficiency possible. Optimum delivery schedules to farms on the laterals are compiled and adjusted, thus allowing the scheduling of water deliveries throughout the entire distribution and storage system to make the most effective and efficient use of the total water supply. Printouts of farm delivery schedules on a lateral are utilized by the ditchrider or other operating personnel to enable him to schedule and utilize water in the lateral in an efficient manner.

In most cases, system scheduling can be utilized with a minimum change in present delivery procedures and without additional personnel. However, scheduling provides for the allocation of water for delivery in accordance with the actual and projected crop water use modified by rainfall, cultural practices, delivery system carrying capacity, and field irrigation characteristics which may result in changes in the water delivery systems of some projects. Field irrigation scheduling may be more difficult to implement. A private consultant must be encouraged to provide the service, or the district must hire and train its own personnel. One employee is required for every 5,000 acres under the field scheduling service.

Technology Transfer and Assistance

The Bureau of Reclamation has developed and demonstrated the usefulness and practicality of applying a large-scale research effort to the water management program. The program has been approached from the standpoint of the "irrigators' program," and irrigators have responded readily to the program and have found it highly useful.

Thus far, experience gained in the program shows that the developed irrigation scheduling procedures are worthwhile to the irrigators and beneficial to the Nation. The water users should be encouraged to evaluate the program with emphasis on how it may assist them to improve efficiencies and reduce costs associated with irrigated agriculture. Irrigation scheduling concepts should be integrated into the entire project operation to enable the most optimum operation. Application of improved water management and irrigation scheduling concepts should be considered in each of the following activities:

Transfer of Technology

1. Extended field demonstration and training programs in which Reclamation irrigation operations and management specialists work with the staffs of individual irrigation districts for periods of 4 months to 2 years to institute practices such as onfarm irrigation scheduling and water delivery scheduling

2. Short-term field demonstration and training activities in which Reclamation specialists may be assigned to individual irrigation districts for as long as 4 months to train district management and operations personnel and institute improved systems of water measurement and accounting, improved district water management policies and operating procedures, improved business practices, etc.

3. Information and training workshops held at central locations over several days or weeks to present new technologies and improved practices to irrigation district employees and irrigators

4. Preparation and distribution of informative and instructive media such as technical and nontechnical publications, films, and other media concerning improved water and energy management practices and their benefits

Technical Services and Assistance

1. Diagnostic examination of individual irrigation systems to identify the causes of excessive water use or water losses and to determine the specific remedial measures (i.e., improvements in facilities, management, or operations) that should be taken
2. Evaluation of cost-effectiveness of proposed improvements or practices to increase water use efficiency
3. Improvement of district and onfarm water measurement systems
4. Improvement of district water management policies and operating procedures
5. Improvement of district business practices (e.g., water accounting and billing)
6. Development of district water conservation plans
7. Development of district contingency plans for periods of drought or low flow

Anticipated Program Benefits

As a general assessment, the anticipated benefits of the WMC program have been divided into three categories. All impacts have not been studied, but some benefits have been documented. Generally, trends and studies support the following ideas:

1. Water users. An increase in crop quality and yield is most important. Irrigation scheduling provides for better use of labor and water with fewer restrictions on water deliveries during periods of peak water use. By proper management, the farmer can reduce leaching of soil nitrogen and other soluble plant nutrients.

2. Irrigation district. Scheduling will improve the economic base associated with the irrigation enterprise; an operational program enables better use of reservoir storage and reduces the load on the delivery system during periods of peak water use. With the use of the computer, the district will have the capability of forecasting delivery requirements and computerizing water storage and delivery records. WMC may reduce drainage requirements and associated problems. By controlling field and farm runoff, some of the district's maintenance requirements would be reduced.

3. National. An irrigation management program helps reduce adverse environmental effects from irrigated agriculture, such as salt loading of a river, while improving the utilization of the natural resources, particularly where chemical fertilizer and pesticides are used. Irrigation management improves the economics of irrigated agriculture and provides needed information for planning and operation of irrigation systems.

Costs and Implementation Methods

While the program is initiated primarily as a demonstration effort, the beneficiaries are expected to financially support these programs into operational functions. The results of these efforts are to be applied in the design of new projects or rehabilitation of irrigation systems. With WMC, management control is provided to the farm operation and to the remainder of the system (storage, carriage, and drainage). This management will result in projects that can be planned and built with more confidence and at a lower cost.

Present data and experience indicate that projected costs for a complete WMC program for large irrigated areas will range from $3 to about $7 per acre per year. This will vary depending on the intensity of service provided and if the service is provided by a private consulting firm or an irrigation district. In some situations the field scheduling may be provided by a private consulting firm with the irrigation district providing its own system scheduling techniques. Normally in this situation, few or no additional personnel would be required by a district. Reclamation is confident that the irrigators and the irrigation districts will ultimately assume full financial support. In some instances the Bureau will extend its demonstration program to an operation service under a water user contract or memorandum of agreement. As the irrigation scheduling program gains acceptance and financing from the water users and as costs and benefits are completely documented, the involvement of Federal funds will decline. There will be a continuing limited need for general engineering and research funds for incorporating new technology developments and updating the program concepts.

Under a cooperative agreement, the Bureau has been working with districts that have purchased minicomputers. A computer programmer may be hired by the district to develop operational software unique to that district. Typical programs that are being developed under this concept include:

1. Payroll
2. Paying and receiving
3. Water accounts and billing
4. Cost accounting
5. Water management; i.e., field irrigation scheduling and distribution system scheduling

The Bureau feels that the overall operation and maintenance of the district will be made more efficient by utilizing the latest data processing techniques. Also, as present district data processing equipment becomes inoperative, replacement with a minicomputer looks more attractive. As WMC gains acceptance and financing from water users increases, the level of Federal funding will be reduced. There will be a continued need for some general research type funding for incorporating new technology and program ideas.

Technical Discussion

The most important phase of the WMC program is to optimize the delivery of water throughout a project's distribution and storage system. This phase, referred to as system scheduling, can be enhanced by improving onfarm water management by increasing irrigation efficiencies through the proper scheduling of irrigations.

Field Irrigation Scheduling

In one method of scheduling irrigations, a daily water budget analysis is made using the daily evapotranspiration rate and amounts of effective irrigation and precipitation. The water budget computes the daily soil moisture depletion by adding the previous depletion and daily evapotranspiration and subtracting irrigation and precipitation. The daily depletion obtained is the amount of water required to refill the soil profile on that day. By projecting the evapotranspiration rate into the future, one can pinpoint the date when the soil moisture depletion will reach an optimum level. The irrigator can be advised when this date will occur and the quantity of water that must be applied to refill the soil profile. The recommended water requirement can be adjusted by the irrigation efficiency so the irrigator can achieve proper application.

The evapotranspiration rate of any given crop will affect the rate of depletion of soil moisture differently throughout the season depending upon prevailing climatic conditions and development of the crop root zone. Thus irrigation intervals will vary throughout the growing season.

Various procedures are available for determining the evapotranspiration rate (consumptive use) of a specific crop. The one procedure most often used in the Reclamation's irrigation scheduling phase of the WMC program was developed by the Agricultural Research Service.

This procedure is based upon determining the evapotranspiration rate for a reference crop such as healthy alfalfa with 10 to 12 inches of top growth. This reference evapotranspiration rate is called the "potential evapotranspiration rate." The evapotranspiration rate of any other specific crop is then determined by adjusting the potential evapotranspiration rate by a crop coefficient. The crop coefficient of a specific crop varies with its stage of growth. Data required to calculate the potential evapotranspiration rate and the evapotranspiration rate of a specific crop consist of climatic data and crop data. The climatic data include the daily maximum temperature,daily minimum temperature, humidity, wind travel, and solar radiation. These data are collected daily at a weather station established at a representative location within the irrigation project. Crop data required consist of crop type and variety, planting dates, emergency date, date of peak consumptive use (referred to as "cover date"), and harvest date. These data are gathered from area experimental station personnel, seed suppliers, and the irrigator himself.

Each week, field probing is used to make a manual determination of the depletion of the soil moisture reservoir using the "feel" method. This depletion is compared to compute depletion, to update the water budget analysis, and to arrive at the final recommendation made to the irrigator. Field personnel make weekly visits to the cooperator to deliver a computer printout, make the irrigation recommendation, and advise the irrigator on any water management problems. Adjustments may be made in the scheduled irrigation date and the amount of water to apply if the total farm delivery requirement exceeds the capacity of the onfarm distribution system. Similar adjustments can also be made if there are conflicts with any onfarm cultural practices. Ground-water information is also collected in areas where the ground water is high enough to interfere with the crop root zone. The information collected from water-table readings gives indications of the time that irrigation intervals may be extended. After a season's irrigation and near harvest time, final soil moisture samples are taken to correlate field and computer depletions.

Another approach for determining consumptive use is the neutron moisture gage. The gage is a proven and a practical field tool for making rapid, accurate measurements of soil moisture. The direct measurement of soil moisture makes quick, accurate determinations of projected irrigation dates and amounts of water to apply; extensive computations are not required to determine evapotranspiration rates. Gage readings taken biweekly in the summer and weekly in the winter provide sufficient information to determine the rate and amount of moisture depletion in the soil. The readings can be plotted on a field graph or the readings can be fed into a computer for analysis. Using either a graph or a computer, the irrigation date and the amount of water to be applied can be scheduled a week in advance with good accuracy.

Distribution System Scheduling

The scheduling of water to fields for optimum crop yields takes into consideration the constraints and limitations imposed by the farm delivery system as well as those of the project distribution system. The compilation and organization of water delivery schedules for the farms on a lateral are referred to as system scheduling. Optimum delivery schedules to farms on the laterals are compiled and adjusted; this allows the scheduling of water deliveries throughout the entire distribution and storage system to make the most effective efficient use of the total water supply.

Printouts of onfarm delivery schedules on a lateral are utilized by the ditchrider or other operating personnel to enable him to schedule and utilize water in the lateral in an efficient manner.

Three levels of accuracy and intensity in determining forecast irrigation requirements are recognized for system scheduling. They are based upon the intensity of field scheduling in use.

Level I, the least intense level, simply determines a system delivery schedule for actual water orders received from irrigators. No future requirements are projected. The length of the timeframe covered is governed by the ordering lead time required by the irrigation district.

Level II determines a system schedule using actual water orders and projected requirements based upon theoretical computations of consumptive use. No visits are made to the fields to verify consumptive use and field moisture status. (Fields involved are not receiving WMC field scheduling services.)

Level III, the most accurate level, involves scheduling actual water orders and projected requirements of fields receiving WMC service. Future requirements can be accurately projected a week in advance.

Ideally, Level III would be the most desired level of system scheduling when a WMC Program is in operation. However, WMC does not usually provide service to all fields within an area where system scheduling is desired. Level II scheduling can then be utilized for all fields not receiving WMC service provided personnel are available to gather necessary crop and soil data and computer storage space is sufficient to hold the extra data. The Gravity System Scheduling Program (GSSP) utilizes Level II and Level III scheduling.

Level I scheduling has been incorporated into a modification of the GSSP. This program was developed to initiate system scheduling over a large area where time, personnel, and computer data storage space were inadequate for Level II scheduling and fields were not receiving WMC service. This program is currently used by an irrigation district in the Southwest.

System scheduling computer programs have been developed to accommodate gravity and sprinkler irrigation systems. The system scheduling programs use the forecasted farm irrigation events to determine the optimum delivery demand at each turnout. Adjustment constraints are imposed by the capacity of the distribution system. Adjustments may be made to maintain the most uniform flow possible throughout the system and to reduce operational waste. Daily flow requirements for each turnout are adjusted as necessary and totaled to determine the lateral diversion requirement and its demand on the main distribution system.

The speed and data storage capabilities of modern computers are utilized to make the numerous calculations required by the WMC Program. However, large centralized computer systems are not required. Recent improvements in computer technology and modifications to the WMC computer programs enable many irrigation districts to use their own "in-house" equipment. This is encouraged under the objectives of the Bureau's WMC Program.

Ideally, the computer can entirely schedule irrigations; tabulation of the delivery requirements by field coordination is desirable and must continue. Field personnel are necessary to assist the irrigators in maintaining effective onfarm water management programs and to coordinate field irrigation schedules with the district operating personnel to optimize water deliveries. Thus, the WMC Program is integrated into the regular, ongoing operation and maintenance program of the water user's district.

Water Delivery Accounting

Computerized water delivery accounting is a logical companion to field irrigation and system scheduling. Both field irrigation scheduling and system scheduling on the computer require irrigation application data. A companion computerized Water Delivery Accounting Program would not only handle water accounting, but it would also make water delivery data available for use by field irrigation and scheduling programs. Such a program has been developed for use on a minicomputer. It has been in operation for 4 years.

Services Provided

At present, the program can provide the irrigator and the district assistance using various levels of irrigation scheduling and systems scheduling. These levels vary with the intensity of the service provided, but all delegate to the irrigator and the district the final decisions of water applications and deliveries. The levels of irrigation scheduling consist of the intense "Field Irrigation Scheduling" and the less intense "Irrigation Guide." The levels of system scheduling consist of the intense "Gravity" or "Sprinkler" programs and the less intense "Consumptive Use" program. The following levels of assistance are described as presently developed. Any of these levels or intensities can be tailored to meet the needs of the irrigators of districts.

1. Irrigation Guide. The Irrigation Guide (exhibit A) gives irrigation intervals for principal crops in an area based on daily evapotranspiration rates and average waterholding capacities for several soils in the area. The guide is updated weekly with daily climatic data from a central location in the area. It gives the average daily water use and the total water use for the next week. This information represents a crop with either an early, average, or late planting date for each farm and a stage of growth that is general for the update period in the area. The Irrigation Guide is proving very effective as a supplemental service. Where the data base and experience enable accurate definitions of general conditions of the area, the recommended irrigation intervals are quite accurate.

2. Field Irrigation Schedule. The Field Irrigation Schedule (exhibits B and C) provides the irrigator or the farm manager with the daily soil moisture status of each of his fields in the program. It gives him recommended optimum dates and the amount of water to apply at the next irrigations. If adequate input data are available, application rates and set times can be included. Because the data requirements are substantially

Exhibit A

EXPLANATION OF COMPUTER PRINTOUT TERMS

This week continues our explanation of computer printout terms and should be saved for future reference.

PLANTS USE THIS AMOUNT OF WATER

AVE. DAILY PLANT WATER USE	This is the average daily amount of water, in inches, that the crop has been using each day during the past week.
AMOUNT WATER USED BETWEEN IRRIGATION	After completion of an irrigation, this should be the amount of water, in acre-inches, available for use by this particular crop before the next irrigation is necessary. We assume in the figure given that the last irrigation filled the soil to capacity.
YOUR LAST IRRIGATION	Enter here the date of last irrigation for any particular crop shown on the list.
IRRIGATION INTERVAL DAYS	This is the recommended days between the last irrigation and the irrigation that is approaching within the next 5 days. This interval is computed from actual past weather and crop conditions and a 5-day future forecast.
YOUR NEXT IRRIGATION	This date is determined by adding YOUR LAST IRRIGATION plus IRRIGATION INTERVAL DAYS for any particular crop shown.

APPLY THIS AMOUNT OF WATER

AMOUNT PER ACRE (ACRE-INCHES)	It is suggested that this amount of water be applied the NEXT IRRIGATION DATE. It is assumed that 65% of the water applied remains in the soil, which is considered to be a good irrigation application efficiency.
MINER INCHES PER 12-HR.SET ACRE	This column is an estimate of the size of the stream necessary to complete an irrigation for a given crop. Multiply this figure by number of acres you plan to irrigate per set. If 24-hour set time is used, divide this by 2.

IRRIGATION GUIDE*

A&B IRRIGATION DISTRICT

COMPUTATION DATE MAY 28, 1974

SILT LOAM SOIL
****

	PLANTS USE THIS AMOUNT OF WATER		REPLACE WATER WITH GOOD IRRIGATION PRACTICES			APPLY THIS AMOUNT OF WATER	
CROPS	AVE. DAILY PLANT WATER USE	AMOUNT WATER USED BETWEEN IRRIGATION	YOUR LAST IRRIGATION	IRRIGATION INTERVAL DAYS	YOUR NEXT IRRIGATION	AMOUNT PER ACRE (ACRE-INCHES)	MINER INCHES PER 12-HR.SET ACRE
Alfalfa	0.24	4.0		15		5.7	24.0
Grain	0.19	2.8		13		4.4	18.5
Sug.Bts.	0.06	1.6		19		2.7	11.3
Potatoes	0.05	1.1		17		1.6	7.1
Beans	0.01	1.2		23		1.9	8.0

12HR.SET/2=24HR.SET

* This information is intended as a "guide only" and must be applied to each individual's crop, soil & water conditions. A detailed irrigation scheduling service is available by contacting the A&B District Office - Ph-436-3152

Exhibit B

```
                         FIELD IRRIGATION SCHEDULE

             Farm No. 13     Mr. I. M. Nobody     Date:  Sep 7
                             Route # 1
                             Somewhere,
                             Alaska  99911

                            AVE    USE      TO-       DAYS        LAST       NEXT
                      RT    DAY    BETWN    DAYS      INTERVAL    IRR      IRRIGATION
Field-Crop-Acres      ZN    USE    IRR      DEPL    PCT-REFILLED  DATE     DATE     AMT
----------------      FT    INS    INS      INS      50 75 100    ----     ----     INS

1 Sorghum     10      3.0   .29    2.0      1.9                   Aug 30   Sep  7   3.3
2 Sorghum     10      3.0   .29    2.0      1.6*      3  5   7    Sep  1   Sep  8*  4.5
3 Sorghum     10      3.0   .44    2.0      3.5#                  Aug 30   Sep  7*  4.4
4 Sorghum     10      3.0   .17    2.0      1.0                   Sep  2   Sep 14   2.3
5 Sorghum     10      3.0   .26    2.0      1.5                   Sep  3   Sep  8   3.3
6 Sorghum     10      3.0   .29    2.0      6.5$      4  6   8                      3.1

                                                   * field has not been checked
                                                   # includes carryover depletion
                                                   $ accumulated irr.req. to date
```

Field Irrigation Schedule showing crop planting data, crop name, field size in acres (2.47 acres = 1 hectare), root zone depth in feet (1 foot = 30.48 centimeters), average daily water use in inches (1 inch = 2.54 centimeters), water use between irrigations in inches, today's soil water depletion in inches, days interval between irrigations depending on percent refill of soil water reservoir, last irrigation date, next irrigation date and amount to apply in inches.

EXHIBIT C

FIELD IRRIGATION SCHEDULE

NEUTRON PROBE

820003 John Doe Ave XYZ Place AZ 80000

CONT/ FIELD	CROP	TURNOUT	FIELD I.D.	AVERAGE DAY USE	ACRES	--NEXT IRRIGATION--		
						DATE	INCHES	ACRE-FEET
820104	ALFALFA	WM17.0-0.6-0.5R	4-B	.19	100.5	1223	3.8	32.16
820204	ALFALFA	WM17.0-3/0L	1-B	.15	100.5	1224	5.5	46.23
820304	ALFALFA	WM17.0-0.6-0.0.6R	5-A	.17	61.1	1223	5.7	29.33

**** - NOT DUE FOR IRRIGATION

** - OVERDUE FOR IRRIGATION

greater for this level than in the Irrigation Guide level, one field technician is needed for every 5,000 to 10,000 acres served by this program. Techniques and equipment being developed will ultimately expand the area that one man can serve. This service can be provided with either consumptive use computer programs or the Neutron Moisture gage.

The gage is being used in several areas to provide the most accurate WMC Program. The gage gives the most accurate determination of soil moisture without the inherent uncertainties of computer programs. Other equipment being used in the WMC Program includes the "Oakfield" probe, tensionmeters, and "Speedy" soil moisture testers. Some or all of these instruments are used by a field technician to assure that pro-jected irrigation dates and amounts generated by the computer are accurate.

The Field Irrigation Schedule Program provides the irrigator with a computer printout once or twice weekly; this is considered an "update" of his irrigation program. This update or schedule incorporates the measured climatic parameters or current soil moisture status measured during the update period. The update period is the elapsed time since the irrigator's last schedule. Evapotranspiration potential is forecasted and updated with each irrigation schedule. This forecasted evapotranspiration potential is based on long-term, historical data and can be modified by a short-term (5 days or less) adjustment for anticipated climatological conditions during the forecast period. Along with the climatic data is a listing of the various fields that would make up a farm or operational unit. Data that could be included in this list are the soil moisture status for each field at the end of the update period, the effective root zone and the water-holding capacity for that zone, and the recommended allowable depletion (items vary depending on the need and preferences of individual districts). These data are followed by the date of the last irrigation and the recommended date for the next irrigation on each field.

3. District-wide Irrigation Requirement. The District-wide Irrigation Requirement Program was developed to assist an irrigation district in tabulating the past, present, and future water demands based on the current cropping patterns and climate (exhibit C). All or a portion of the district deliveries to date, crop consumptive use to date, irrigation requirement to date, and forecast demand can be determined. Irrigation requirements differ from the crop consumptive use in that it reflects the efficiency of the system (farm and canals) and any effective rainfall.

The district's present delivery records and cropping patterns are used in addition to the consumptive use and climate data available from any field scheduling program. The information can be tabulated for any portion of the distribution system and for the entire district. This program will assist the operating personnel of the district in the operation of the distribution and storage facilities.

The Gravity System Scheduling Program was developed to assist irrigation districts that primarily deliver water to gravity irrigation systems.

Exhibit D

DISTRICT-WIDE IRRIGATION REQUIREMENT SHOWING WATER DEMANDS TO DATE AND FORECAST CONSUMPTIVE USE FOR THE NEXT MONTH

Units = Acre-feet

```
 DATE  JUL 22  1977          AREA  OAKS

--------- IDENTIFICATION ---------  -------- HISTORY --------             -- CROP CONSUMPTIVE USE--

                            BASE      MEAS    CU TO-   IRR. REQ.   TODAYS                  NEXT     NEXT
 BLOCK      CROP    ACRES   ALLOC.    DEL     DATE     TO-DATE   IRR. REQ.   TOMMORROW     WEEK     MONTH

BLOCK 1    CORN-E    1200    3000             1490.      912.      25.5         26.2       172.     432.

BLOCK 1    CORN-M    1200    3000             1380.      865.      25.7         26.4       176.     444.

BLOCK 1    CORN-L    1200    3000             1197.      740.      26.0         26.4       182.     589.

BLOCK 1    GRAIN     1200    3000             1644.     1008.      23.1         21.1        87.     250.

BLOCK 1    ALFALFA   1200    3000             2179.     1117.      23.1         23.9       164.     606.

BLOCK 1    *TOTAL*   6000   15000   10500.    7890.     4643.     123.4        124.1       781.    2320.


              TOTAL    PERCENT OF
             TO-DATE    DELIVERY

DELIVERIES   10500.      100.      BASE ALLOCATIONS =   15000 ----- DELIVERIES TO-DATE = 70.0 PERCENT
C.U. CROP     7890.       75.      SEASONAL C.U.    =  11687. ----- SUM TO-DATE        = 67.5 PERCENT
I.R. CROP     4643.       44.      EFFECTIVE PRECIP =   3247. ----- PERCENT OF C.U.    = 41.2 PERCENT
```

The program uses actual water orders received from the irrigators and forecasted irrigation events developed through field irrigation scheduling (exhibit E). The frequency and accuracy in scheduling field irrigations are reflected in the scheduling of the distribution system.

At each turnout the actual water orders received from the irrigators are tabulated. Beyond the confirmed water orders, the delivery requirement for each turnout is computed on the predicted demand and the maximum capacity of the turnout. Daily flow requirements for each turnout are totaled to determine the demand on the lateral and main distribution system. With this information, the operating personnel of the district can make the necessary adjustments to meet the requirements and requests of the irrigators.

Program Availability

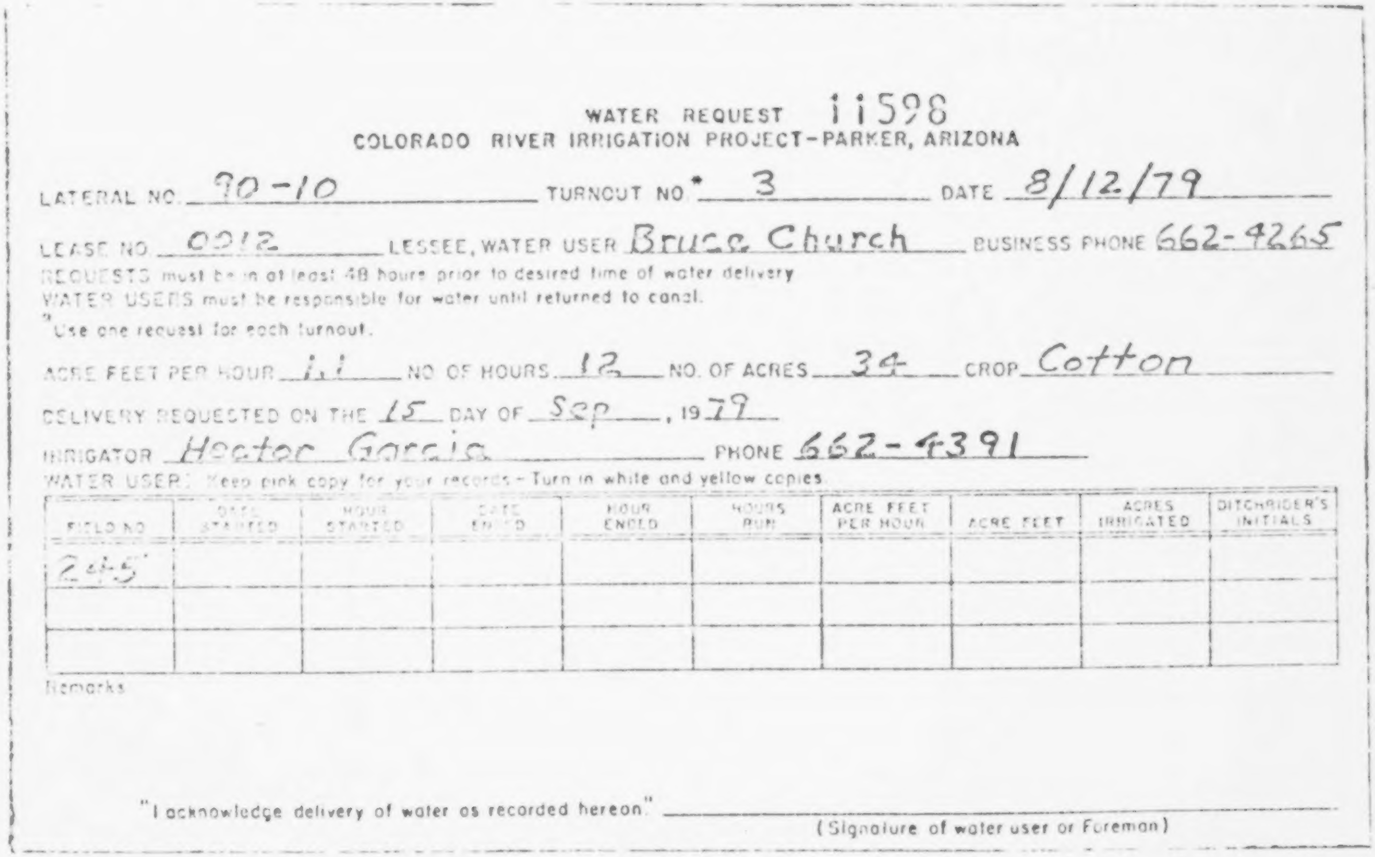
All of the programs described above are available for district implementation. Most of the computer packages discussed are available on the Bureau's central computer in Denver. Many of the programs have been adapted to small in-house computers used by many irrigation districts. Other programs for small computers have been designed to meet specific needs of individual districts but are easily adaptable to almost any district's operational procedures.

Exhibit E

SYSTEM SCHEDULING FOR A LATERAL ON A GRAVITY IRRIGATION SYSTEM
SHOWING THE ORDERED CFS REQUIREMENTS AND FORECAST CFS REQUIREMENTS

LATERAL W-11.7 DATE 801

	WATER ORDERS						FORECAST TURNOUT SCHEDULE								7-DAY SUB-TOTAL
	731		801		802		803		804		805		806		807		
TURNOUT	CFS	HRS	CFS	HRS	CFS	HRS	CFS	HRS	CFS	HRS	CFS	HRS	CFS	HRS	CFS	HRS	AC-FT
0.5L					15.	30.											37.1
0.6R	7.	24.															
0.8L			15.	36.											15.	15.	63.0
1.1R							15.	7.					15.	10.			20.3
1.1L					12.	36.					15.	41.					86.5
1.2R									15.	16.							19.2
1.5R			12.	12.			15.	10.									24.7
1.5L																	
2.0R																	
2.0L	8.	24.							15.	7.							6.6
DAILY ACFT			41.6		[illegible]		40.4		27.8		23.7		33.1		18.5		253.4
60 CFS.DAY			20.72		24.16		20.30		11.88		14.85		16.57		9.27		13.



WATER REQUEST 11598

COLORADO RIVER IRRIGATION PROJECT–PARKER, ARIZONA

LATERAL NO. 90-10 TURNOUT NO.* 3 DATE 8/12/79

LEASE NO. 0012 LESSEE, WATER USER Bruce Church BUSINESS PHONE 662-4265

REQUESTS must be in at least 48 hours prior to desired time of water delivery

WATER USERS must be responsible for water until returned to canal.

*Use one request for each turnout.

ACRE FEET PER HOUR 1.1 NO. OF HOURS 12 NO. OF ACRES 34 CROP Cotton

DELIVERY REQUESTED ON THE 15 DAY OF Sep, 1979

IRRIGATOR Hector Garcia PHONE 662-4391

WATER USER: Keep pink copy for your records – Turn in white and yellow copies.

FIELD NO.	DATE STARTED	HOUR STARTED	DATE ENDED	HOUR ENDED	HOURS RUN	ACRE FEET PER HOUR	ACRE FEET	ACRES IRRIGATED	DITCHRIDER'S INITIALS
245									

Remarks

"I acknowledge delivery of water as recorded hereon." ______________________

(Signature of water user or Foreman)

Example 1. Water Request Submitted by Farmer.

```
DIVISION NO: 2
DATE: 5 / 10 / 79
PAGE 1

----------------------------------------------------------------------------------------------------------------
                                  !      !          WATER REQUESTS         !              FORECAST DELIVERIES             !
                                  !      !           !           !           !           !           !           !           !
LAT/ ORDER                        !      !    509    !    510    !    511    !    512    !    513    !    514    !    515    !    516    !
TO# NO.    FARM                   ! FIELD!AF/HR HRS!AF/HR HRS!AF/HR HRS!AF/HR HRS!AF/HR HRS!AF/HR HRS!AF/HR HRS!AF/HR HRS!
----------------------------------------------------------------------------------------------------------------

27R              CAPACITY= 290 CFS
 37  3628  RICHARD LEIVAS         ! N-2  ! 1.4 13# !           !           !           !           !           !           !           !
 39  3630  RICHARD LEIVAS         ! N-3  ! 1.4 17# !           !           !           !           !           !           !           !
 41  3631  RICHARD LEIVAS         ! N-4  !           ! 1.4 30*  !           !           !           !           !           !           !
 42  1825  TONY MARTINEZ          !  42  ! 1.4 8 #  !           !           !           !           !           !           !           !
 44  1826  TONY MARTINEZ          !  44  ! 1.4 8 #  !           !           !           !           !           !           !           !
 46  1827  TONY MARTINEZ          !  46  ! 1.4 8 #  !           !           !           !           !           !           !           !
 47  2078  JOE HERNANDEZ          !  47  !           ! 1.1 24*  !           !           !           !           !           !           !
 49  2079  JOE HERNANDEZ          !  48  !           !           ! 1.1 24*  !           !           !           !           !           !
 53  0     JOHN C LEIVAS          !  50  !           !           !           ! 1.7 3<   !           !           !           !           !
 52  0     JESSE SALCEDO          !  52  !           !           !           ! 1.4 4<   !           !           !           !           !
 54  4091  ADRA FARMS             ! 14   !           !           ! 1.2 10*  !           !           !           !           !           !
 55  4092  ADRA FARMS             ! 15N  !           !           !           ! 1.2 12*  !           !           !           !           !
 59  0     DANIEL LEIVAS          !  59  !           !           !           ! 1.4 11<  !           !           !           !           !
----------------------------------------------------------------------------------------------------------------
   DAILY AC-FT                    !      !    138    !    112    !     67    !    120    !     47    !      0    !     63    !      9    !
   AVERAGE DAILY FLOW, CFS        !      !     69    !     57    !     34    !     61    !     24    !      0    !     33    !      5    !


27R-25           CAPACITY= 75 CFS
  3  0     MIKE MARTINEZ          ! 13   ! 1.5 12*  !           !           !           !           !           !           !           !
  5  0     CECILIA MARISCAL       !   5  !           !           !           ! 1   2<   !           !           !           !           !
  6  0     CECILIA MARISCAL       !   6  !           !           !           ! 1.3 7<   !           !           !           !           !
  8  0     CECILIA MARISCAL       !   8  !           !           !           ! 1.3 3<   !           !           !           !           !
 15  0     LUDWIG FARMS           !   1  !           !           !           !           ! 1.1 20   !           !           !           !
 16  0     CECILIA MARISCAL       !  16  !           !           !           ! 1   6<   !           !           !           !           !
 19  0     BOBBY HOLT             !  19  !           !           !           ! 1.8 2<   !           !           !           !           !
 20  0     MIKE MARTINEZ          ! 21   !           !           !           !           !           !           ! 1.5 7    !           !
 21  0     BOBBY HOLT             !  21  !           !           !           !           ! 1.5 2    !           !           !           !
 26  0     BERNAL FARMS           ! H 2  !           !           !           !           !           !           !           ! 1.5 6    !
 29  0     ADRA FARMS             ! 16S  !           !           !           ! 1.2 3<   !           !           !           !           !
 34  4193  ADRA FARMS             ! 31   !           ! 1.2 24*  !           !           !           !           !           !           !
----------------------------------------------------------------------------------------------------------------
   DAILY AC-FT                    !      !     18    !     29    !      0    !     32    !     25    !      0    !     11    !      9    !
   AVERAGE DAILY FLOW, CFS        !      !      9    !     15    !      0    !     16    !     13    !      0    !      6    !      5    !


27R-29           CAPACITY= 55 CFS
  2  0     TONY MARTINEZ          !   2  !           !           !           ! 1.6 8<   !           !           !           !           !
----------------------------------------------------------------------------------------------------------------
   DAILY AC-FT                    !      !      0    !      0    !      0    !     13    !      0    !      0    !      0    !      0    !
   AVERAGE DAILY FLOW, CFS        !      !      0    !      0    !      0    !      7    !      0    !      0    !      0    !      0    !

 * - FIRM WATER ORDER    # - OVERDUE WATER ORDER    < - CV.POOL FORECAST IRRIGATION
```

Example 2. Irrigation System Delivery Schedule.

```
DIVISION NO: 2
DATE: 5 / 10 / 79
PAGE 2

------------------------------------------------------------------------------------------------------------------------
                                       I     I          WATER REQUESTS          I                  FORECAST DELIVERIES                     I
                                       I     I          I          I            I          I          I          I          I              I
LAT/ ORDER                             I     I    509   I    510   I    511     I    512   I    513   I    514   I    515   I    516       I
TOT  NO.      FARM                     I FIELDIAF/HR HRSIAF/HR HRSIAF/HR HRSIAF/HR HRSIAF/HR HRSIAF/HR HRSIAF/HR HRSIAF/HR HRSI
------------------------------------------------------------------------------------------------------------------------

27R-36            CAPACITY= 40 CFS
  5  0      BERNAL FARMS               I H 7  I          I          I          I          I          I          I 1.4 9    I          I
  7  4684   BERNAL FARMS               I H 4  I          I          I          I          I          I          I 1.4 10   I          I
 11  0      BERNAL FARMS               I H 5  I 1.5 20#  I          I          I          I          I          I          I          I
 16  0      BERNAL FARMS               I H 9  I          I          I          I          I 2   11   I          I          I          I
 18  0      JIM JENNINGS               I  18N I          I          I          I 1.3 8<   I          I          I          I          I
 18  0      JIM JENNINGS               I  18S I          I          I          I 1.3 15<  I          I          I          I          I
 24  0      JIM JENNINGS               I  24  I          I          I          I          I          I          I 1.9 3    I          I
 25  4782   HURTADO FARM               I  25  I 1   60#  I          I          I          I          I          I          I          I
------------------------------------------------------------------------------------------------------------------------
   DAILY AC-FT                         I      I    54    I    24    I    12    I    35    I    22    I    0     I    52    I    0     I
   AVERAGE DAILY FLOW, CFS             I      I    27    I    12    I    6     I    18    I    11    I    0     I    27    I    0     I

27R-36 -10        CAPACITY= 40 CFS
  2  0      JIM JENNINGS               I   2  I          I          I          I 1.7 3<   I          I          I          I          I
  3  0      JOHN HILL                  I   3  I          I          I          I          I          I          I 1.7 3    I          I
------------------------------------------------------------------------------------------------------------------------
   DAILY AC-FT                         I      I    0     I    0     I    0     I    5     I    0     I    0     I    5     I    0     I
   AVERAGE DAILY FLOW, CFS             I      I    0     I    0     I    0     I    3     I    0     I    0     I    3     I    0     I

 * - FIRM WATER ORDER   # - OVERDUE WATER ORDER   < - OVERDUE FORECAST IRRIGATION
```

Example 2 (continued). Irrigation System Delivery Schedule.

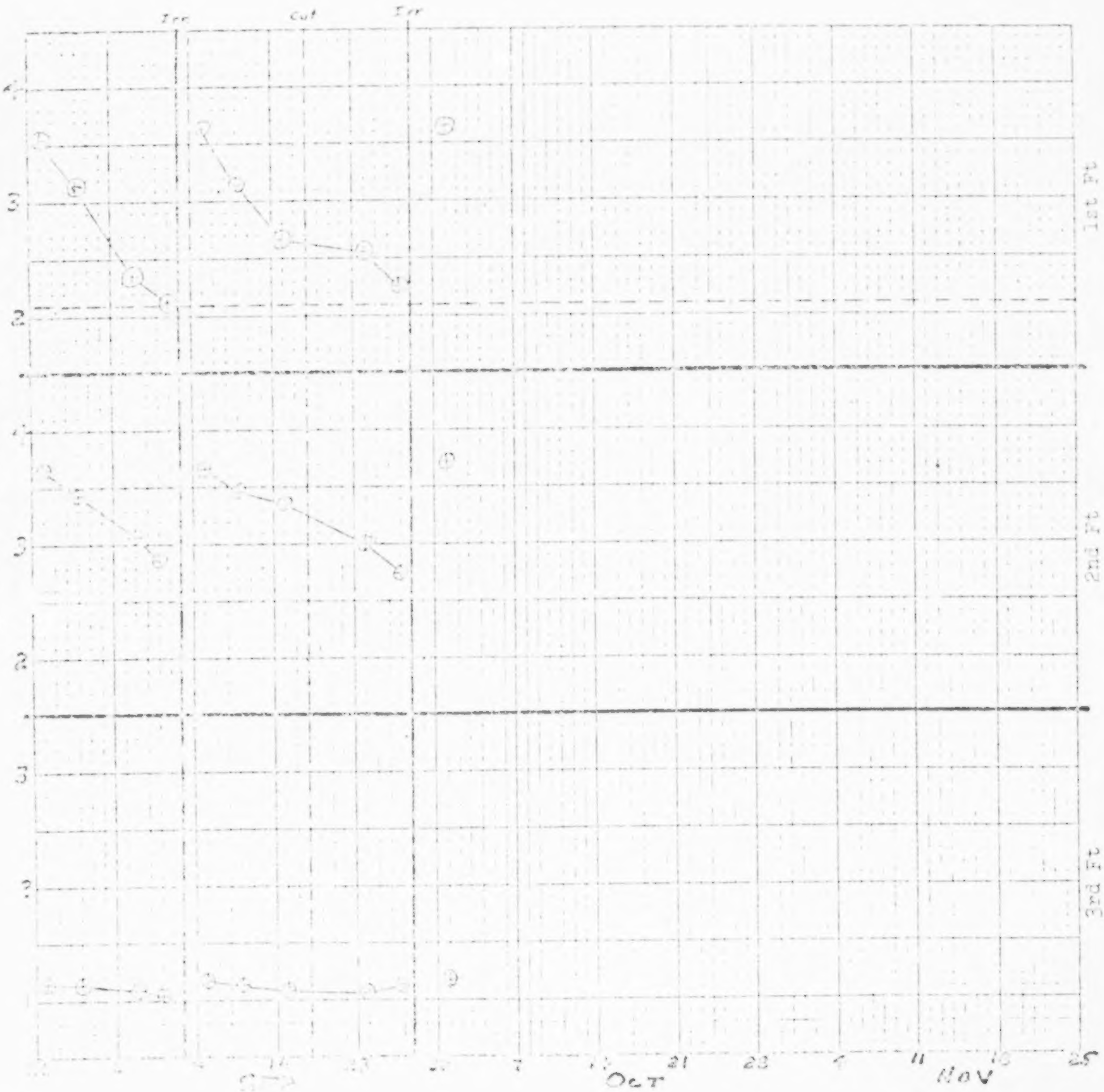
DIVISION NO. 5

DATE 9/ 19/ 79
PAGE 1

THE FOLLOWING REQUESTS ARE OVERDUE FOR DELIVERY OR COMPLETED CARDS HAVE NOT BEEN TURNED IN. CARDS FOR COMPLETED DELIVERIES SHOULD BE TURNED IN IMMEDIATELY. OVERDUE REQUESTS SHOULD BE GIVEN FIRST PRIORITY.

REQUEST NUMBER	NAME	LATERAL	TURN OUT	DATE REQ'D	AF/HR REQ'D	HOURS REQ'D
6143	Leuppe, Frank	73 -25R- 3	3	915	1.2	7
6144	Leuppe, Frank	73 -25R- 3	5	915	1.3	7
6145	Leuppe, Frank	73 -25R- 3	9	916	1.2	7
8456	Chrismer Farms, Inc.	73 -25R- 3	23	915	1.2	15
8457	Chrismer Farms, Inc.	73 -25R- 3	22	916	1.2	15
9226	Double J Farms	73 -25R	33	915	1.5	12
9227	Double J Farms	73 -25R-26	1	916	1.5	10
9228	Double J Farms	73 -25R-23	3	916	1.5	10
9237	Double J Farms	73 -25R-15- 9	2	915	1.5	14

Example 3. Overdue and/or Missing Water Order Report - Updated Daily.



PROBE	DATE	6"	18"	30"
	824	-Irr		
4	827	8130	8290	3050
4	830	7280	7810	3030
4	904	5260	7080	2970
4	906	5090	6630	2810
4	910	8210	8350	3110
4	913	7170	7900	2990
4	917	6210	7650	2970
4	924	6090	6940	2950
4	927	5370	6330	3030
4	1001	8320	8430	3070

CROP Alfalfa REFILL 2.1
PLANT DATE
FARM OPERATOR Aspa Farms (69)
FIELD IDENT 36 West (70)

IRRIGATION MANAGEMENT SERVICE
P O BOX 2036
POSTON, AZ 85371
662-4358

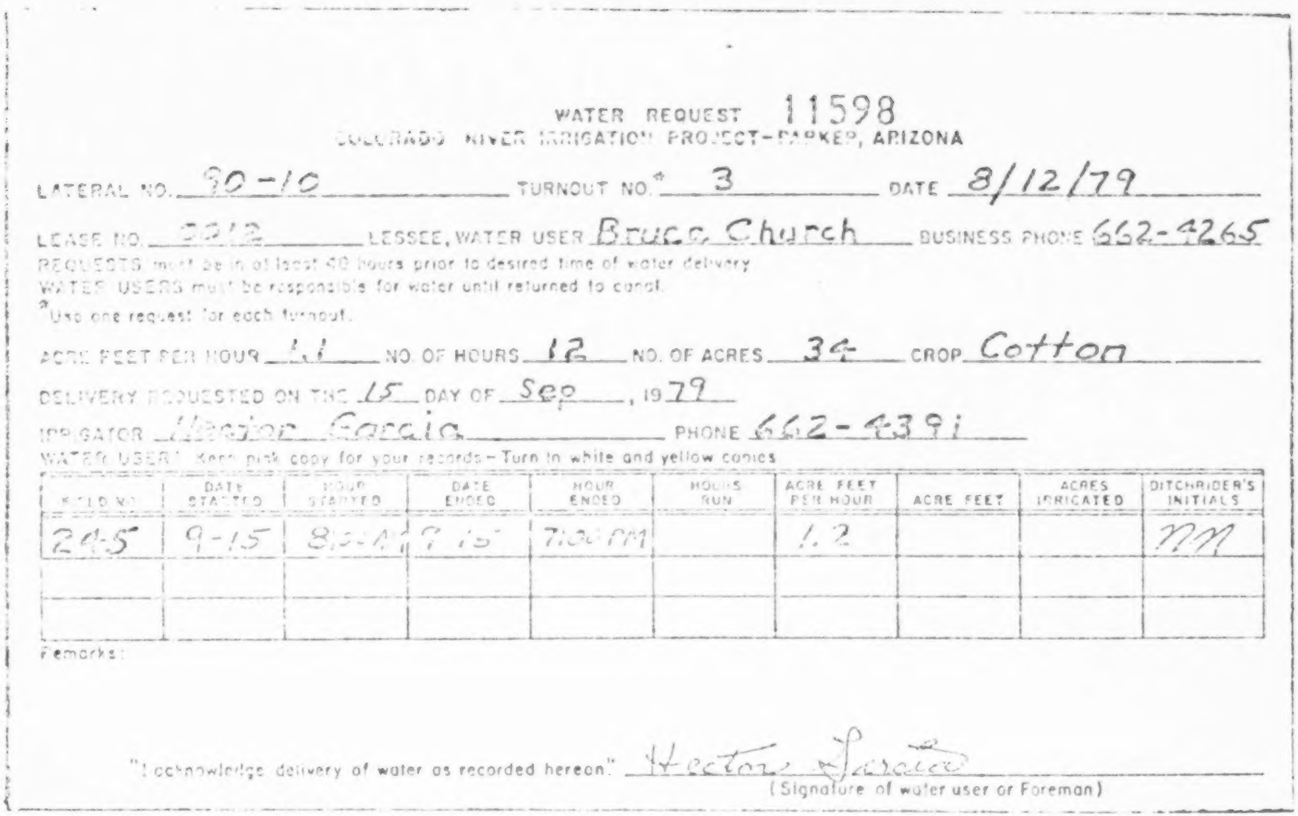
DATE: APR 26

CRIT FARMS
P O BOX 1028
PARKER AZ 85344

FIELD, CROP AND ACRES			IRRIGATIONS					
			NEXT			LAST		
				AMT			AMT	
			DATE	IN.	AC-FT	DATE	IN.	AC-FT
12	ALF	38	5/ 3	5.7	18	4/19	4.2	13.3
16	ALF	37	5/ 3	5.7	17.6	4/19	5.4	16.7
17	ALF	37	5/ 9	5.7	17.6	4/21	3	9.3
21	ALF	36	4/29	4	12	4/16	0	0
22	ALF	37	5/ 9	5.6	15.4	4/21	6.8	21
26	ALF	36	5/ 7	5.6	16.8	4/16	11.9	35.7
27	ALF	36	5/ 7	4.2	12.6	4/18	10.8	32.4
30	ALF	35	5/ 7	4	11.9	4/18	8.7	25.4
31	ALF	35	5/ 7	5.3	15.5	4/18	11.1	32.4
41	ALF	38	4/30	4.2	13.2	4/18	6.3	20
45	ALF	37	*****			4/20	0	0
46	ALF	35	5/ 3	4	12.0	4/18	0	0
48	ALF	35	4/29	4.6	13.3	4/18	0	0
49	ALF	38	4/29	3.5	10.9	4/18	0	0
56	ALF	35	4/23*	3.5	10.3	4/14	7.4	21.6
2- 2	ALF	32	4/27	3.4	9.1	4/14	17.3	46.1
2- 3	ALF	25	4/26	5	10.4	4/13	3.2	6.7
2- 8	ALF	16	4/26	3.4	4.2	4/14	5	6.7
2-11	ALF	21	4/25*	5.1	8.9	4/13	4.5	7.9
2-13	ALF	52	4/28	4.2	16.2	4/12	4.2	18.2
2-16	ALF	29	4/27	4.5	10.9	4/10	8.1	19.6
2-17	ALF	30	4/27	4.6	11.5	4/10	3.9	9.8

* - FIELD OVERDUE FOR IRRIGATION
***** -- IRRIGATION NOT REQUIRED BEFORE MAY 10

Example 5. Field Irrigation Schedule - Prepared for Farmer Weekly.



WATER REQUEST 11598

COLORADO RIVER IRRIGATION PROJECT–PARKER, ARIZONA

LATERAL NO. 90-10 TURNOUT NO.* 3 DATE 8/12/79

LEASE NO. 3312 LESSEE, WATER USER Bruce Church BUSINESS PHONE 662-4265

REQUESTS must be in at least 40 hours prior to desired time of water delivery

WATER USERS must be responsible for water until returned to canal.

*Use one request for each turnout.

ACRE FEET PER HOUR 1.1 NO. OF HOURS 12 NO. OF ACRES 34 CROP Cotton

DELIVERY REQUESTED ON THE 15 DAY OF Sep, 1979

IRRIGATOR Hector Garcia PHONE 662-4391

WATER USER: Keep pink copy for your records – Turn in white and yellow copies

FIELD NO.	DATE STARTED	HOUR STARTED	DATE ENDED	HOUR ENDED	HOURS RUN	ACRE FEET PER HOUR	ACRE FEET	ACRES IRRIGATED	DITCHRIDER'S INITIALS
245	9-15	8:00 AM	9-15	7:00 PM		1.2			NN

Remarks:

"I acknowledge delivery of water as recorded hereon." Hector Garcia

(Signature of water user or Foreman)

Example 6. Completed Water Order Returned by Ditchrider.

UNITED STATES
DEPARTMENT OF THE INTERIOR
Bureau of Indian Affairs
Colorado River Irrigation Project
Parker, Arizona

Date: Aug 31

Cotton Tail Farms
Rt. 1, Box 48-Y
Parker AZ 85344

LESSEE NO. 0015

ORDER NO.	CROP/	ACRES	LATERAL	TURN OUT	IRRIGATION ***BEGAN** DATE	TIME	FLOW	HOURS	ACRE FEET
*****	********		****************	****	*****	****	****	*****	******
5223	ALF	49	79	18	8/20	300	1.40	18.00	25.2
707	ALF	92	73 -19R	6A	8/ 7	400	1.30	19.00	24.7
708	ALF	10	73 -19R	6A	8/ 7	100	1.30	3.00	3.9
709	ALF	30	73 -19R	6A	8/ 6	1700	1.30	8.00	10.4
5219	PRE	49	79	20	8/12	300	1.50	41.00	61.5
710	ALF	84	79	14	8/11	900	1.40	18.00	25.2
714	ALF	30	73 -19R	6A	8/22	1000	1.30	6.00	7.8
5217	ALF	84	79	14	8/ 1	600	1.50	21.00	31.5
701	ALF	49	79	16	8/ 4	2200	1.40	15.00	21.0
703	ALF	49	79	18	8/ 5	1300	1.40	19.00	26.6
0	ALF	45	79	16	8/ 4	900	1.40	28.00	39.2
704	ALF	49	79	18	8/ 6	800	1.50	16.00	24.0
5218	PRE	38	79	20	8/13	2000	1.40	38.00	53.2
5221	ALF	49	79	16	8/15	1000	1.50	11.00	16.5
5224	ALF	49	79	16	8/18	800	1.40	12.00	16.8
5222	ALF	49	79	18	8/19	900	1.40	18.00	25.2
713	ALF	92	73 -19R	6A	8/24	700	1.80	25.00	45.0

This year's allotment	2666.5
Water used this month	457.7
Allotment remaining	48.1

Example 7.- Water Use Statement - Prepared for each Wateruser Monthly.

Irrigation Data for the month of August - 1979

		Allotment Data			
Lessee	Water User	Year's Allot.	Month's Use	Acres Irri.	Remain. Allot.
0001	Alberty, John	1901.0	364.1	578.0	-75.4
0002	Alcaida, Bill	2901.3	389.0	805.0	-278.4
0004	Baldwin, Bill	3391.0	465.6	995.0	370.0
0005	Bernal, Angelita	806.5	122.2	225.0	205.6
0006	B & B Farms	5577.1	857.0	1358.0	397.6
0011	Chrismer Farms, Inc.	7078.2	1154.9	1930.0	1988.6
0012	Bruce Church, Inc.	34716.0	5505.8	9177.0	12815.3
0013	Colorado River Indian Tribes	453.0	30.9	53.0	201.3
0014	CRIT Farms	22507.1	3227.2	5843.0	2084.8
0015	Cotton Tail Farms	2666.5	457.7	897.0	48.1
0019	Evans, Marlene S.	160.6	.0	.0	160.6
0020	Esquerra, Leandro	401.7	50.6	60.0	-28.9
0021	Francis, O. B.	1326.9	200.9	174.0	-349.4
0024	Gerrard, M. H.	1968.5	20.0	75.0	1338.5
0026	Hanna Farms	1135.5	158.5	350.0	176.2
0027	Hernandez, Joe	1932.5	333.5	719.0	485.3
0028	Hill, David Sr.	1406.2	256.9	204.0	-41.8
0029	Hill, John Sr.	774.2	209.0	153.0	-357.7
0031	Holt, Glen	9467.6	1313.3	2269.0	1636.8
0032	Holt, Margaret	2742.4	723.2	764.0	570.9
0034	Moon Mountain Ranch	5405.5	972.6	2230.0	-476.6
0035	Jennings, James	4600.1	1042.0	1466.0	-260.7
0037	Leivas, Augustine	391.5	35.1	70.0	165.0
0038	Leivas, Henry	15884.7	2013.4	4593.0	5797.4
0039	LePera School	41.5	22.2	25.0	-25.9
0040	Leuppe, Frank	1191.8	80.6	347.0	632.0
0041	Ludwig, Charles	376.5	95.2	144.0	-91.2
0042	Ludwig, Dale	1111.0	198.8	320.0	263.0
0043	M & V Farms	3469.5	953.7	1164.0	-319.6
0044	Mapes Produce	1959.5	275.5	432.0	833.6
0045	Mariscal, Cecelia	1592.8	175.3	374.0	352.6
0046	Martinez, Dick	360.7	68.8	120.0	-32.4
0047	Martinez, Erlinda	3205.6	467.0	586.0	593.9
0048	Martinez, Mike	5254.8	719.4	1154.0	274.7
0049	Martinez, Sam	3342.2	228.8	532.0	1451.4
0050	Martinez, Tony	2933.8	314.9	692.0	456.5
0051	Marentes, Esther	206.7	38.4	76.0	22.3
0053	McGuire, Kennel D.	270.5	24.6	43.0	37.6
0054	McGuire, Thomas	497.0	45.9	116.0	44.0
0056	Nelson Brothers	1113.0	65.0	210.0	694.8
0057	Noroian, Nish	5225.0	1054.7	1945.0	-252.3
0060	Patrick, James	7.0	.0	.0	7.0
0061	Quail Mesa	20001.0	4784.7	11304.0	-3342.9
0062	Rademacher, Richard C.	628.6	57.3	185.0	27.2
0063	Romo, Manuel	603.5	61.0	137.0	215.0
0064	Rovey Land & Cattle Co.	14195.0	2366.4	4028.0	702.8
0066	Salvato, Vence	9279.1	1978.6	3647.0	-205.2
0067	Sharpe, Fred & Ernestine	738.5	105.8	148.0	142.0

Example 8. Water Allotment and Use Summary - Prepared Monthly.

Water Systems Operation and Maintenance Workshop - 1990

WATER SYSTEMS MANAGEMENT, PART 3

Session 2-3

L. Albert Brower

Introduction

The advent of the data processing industry over the last 3 decades has brought us to an environment today where the term "computer" is a household word. We see computers doing inventory and checkout work at the local supermarket, provide accurate and immediate medical analysis at the hospital, and handle intricate engine diagnostics at your favorite automobile dealership. Modern automotive engineering uses microprocessors to automatically adjust engine performance and vehicle operation. For many years, computers have provided an important role in the banking industry and have also printed our paychecks. Obviously, we are married to the computer institution; therefore, we should further our knowledge of how we can use these "power tools" in our work environment.

The objectives of this course are to familiarize you with the work a computer can accomplish in an irrigation district office, how to buy a computer or more specifically a microcomputer, and the meaning of some of the computer terminology.

What is a Microcomputer

There are many names given to the type of computer best suited for the requirements of an irrigation district. Some names are minicomputer, microcomputer, small business computer and personal computer (PC), desktop computer, and workstation. Most of these refer to that part of the computer industry with equipment priced less than $50,000. For this discussion, the term microcomputer is used and refers to a computer small in physical size and not requiring a special office environment such as additional air conditioning or heating. The power requirements for a microcomputer usually meet the existing electrical wiring system. However, a dedicated line for the microcomputer is usually worthwhile, especially if heavy-duty electrical equipment is used nearby.

Generally, a microcomputer consists of a box containing the electronic boards on which silicon microprocessor chips are mounted. Boards are available to perform many different operations, such as enhance the internal memory of the machine, communicate with peripheral devices such as printers or computer networks, communicate with other computers via phone connections, or control input/output functions. This box will also contain the data storage devices, such as hard disks which can contain from 10Mb (Megabytes) to 100+ Mb of information, or floppy diskette drives that can transfer data from 360Kb (Kilobytes) to 1.44Mb floppy diskettes. These floppy diskettes normally are of two physical sizes, either 5-1/4 inch or 3-1/2 inch square. There is a keyboard similar to a typewriter but with a few more special keys, and a monitor that is either monochrome or color. The memory of a modern microcomputer normally begins at the 512Kb level, a minimum requirement for many software packages, and can go up to perhaps 16Mb or more.

In addition, a printer is desirable to produce the output records from the microcomputer. These devices allow many alternatives for print quality and speed. The most common printer is the Dot-matrix which forms characters from a series of dots. Little wires strike the ribbon in specified patterns to form the characters. The impact printer emulates the typewriter design, basically using a wheel containing all the characters that is struck by a hammer against the ribbon. This type of printer is usually slower than the dot-matrix. The laser printer uses laser technology to quickly, and quietly, produce very good quality output. Laser printers that print colors and graphics, and on both sides of the paper, are also available.

The cost of a microcomputer on today's market to fit the needs of the average irrigation district will probably be less than $10,000. Some of the personal (home) computers sold at the local electronics or computer stores, or through mail order sources, begin at a price of $500. Such a microcomputer can be configured to the district's needs by adding increased memory capacity, data storage devices, a high resolution monitor, and a high performance printer, for a total price in the $3,000 to $5,000 range.

What Microcomputers Can Do for an Irrigation District

Most irrigation districts already have some form of automated system for their recordkeeping procedure. Perhaps a bookkeeper uses a calculator and/or a posting machine to total the water orders throughout the irrigation season for billing purposes. The microcomputer can accomplish this task in a fraction of the time, allowing the bookkeeper to do other important work. Frequently, the use of a microcomputer system does not reduce the number of personnel in the office, but it does help accomplish more work in an 8-hour day. An operator is still required to enter the data into the system and execute the commands to produce the desired results. Most microcomputer systems are not difficult to operate as self-training manuals are usually provided by the manufacturer, or training courses may be offered at some cost. There is also training software available to help personnel understand how to run the machine's operation system.

A microcomputer can perform the calculations and bookkeeping requirements of the WMC (Water Management and Conservation) program. Software, consisting of the actual computer programs, is now written and available from the Bureau of Reclamation to place on most IBM/IBM-compatible microcomputers. Depending upon the capabilities of the equipment, this software may have to be modified or rewritten to accommodate the microcomputer specifications. The WMC program, in addition to field-by-field scheduling of irrigations, is capable of providing information on scheduling water throughout the distribution system. Water order data, as provided to the district from the individual farmers, are input to the microcomputer. A data base consisting of the type of crops, acreage, soil types, and layout of the distribution system is also retained within the storage medium. Using this information, the water demands on the distribution system can be calculated and a forecasted demand schedule can be printed.

Using the WMC data base, especially the data associated with water orders, the district can computerize the water accounting procedure to keep updated records on water used by each farmer. Perhaps these data can be checked against the allocated yearly usage figures to flag those areas approaching the allocation. Billing for water used can be quickly achieved from the water accounting records, and the microcomputer can even produce the bill on a special form and also produce the mailing label to send the bill to the water user.

There are many different records and reports irrigation districts retain for their own management. Given the proper type of input data, the microcomputer can complete practically all the record management requirements. Records and reports regarding the water user can be produced containing such data as ownership, legal description, operator, addresses, acreage, etc. Such records can assist with assessments, water user ledgers, stockholders' ledgers, and other reportings from the district to the individual owner-operator. An inventory system can be built into the microcomputer to help keep track of real property and movable equipment. Computer programs can be purchased or written at the local level to aid with determination of equipment depreciation and, of course, the service records for the equipment can be placed into the computer.

Personnel records are a good application for a microcomputer. Packaged programs can be purchased to help maintain time and attendance records on each employee, produce payroll journals, print W-2 forms at year-end, and even print and prepare the mailing of individual paychecks. Financial statements of the districts' operation can be produced by microcomputers, and operation and maintenance cost accounting reports are also a possibility. Most of the reporting requirements can share a common data base within the storage medium, reducing the duplication of effort commonly found in records management. A computer has the added advantage of producing legible and neatly cataloged reports, all of which can aid the occasional auditing procedure.

Word processing is a tool the entire district staff will use to develop everyday correspondence and reports. Once learned, this capability alone can justify the acquisition of a microcomputer system. Many popular word processing software packages are available. Automated "spreadsheet" and "data base" software can also enhance the reporting procedures.

Microcomputers can aid the engineering staff of the district by calculating drainage requirements, sprinkler design problems, and construction details. Graphical analysis is also a potential usage of these types of computer systems.

Another session of the workshop pertains to water systems management using remote and automatic devices to control the entire water delivery system. With a somewhat more sophisticated microcomputer acting as a centralized computer, telemetered data from the control points can be interpreted to perform the complicated control functions. This capability coupled with the WMC (Water Management and Conservation) program, which predicts water demands on the distribution system, can produce a completely automated irrigation system. This can provide accurately managed water delivery systems for the most efficient use of water. Assistance from the Bureau at the project, regional, and Denver Office levels is available to help you choose the right microcomputer system for your district.

How to Choose a Microcomputer - 10 Easy Steps

The next big question is where to begin? Purchasing or leasing a microcomputer is in some regards similar to purchasing a backhoe. First, you must determine what you want it to do, what is available on the market, who can operate it, and of course, the costs involved. Perhaps a lease agreement is the best alternative if you think the microcomputer may become obsolete. Not all companies offer lease or lease-purchase options, but if the options are available they should be economically considered, especially if more powerful equipment is planned. Presently, the microcomputer industry is changing its product lines almost daily, but generally the new products allow more performance power for equal cost.

The 10 steps listed here are possible procedures you may desire to follow as you investigate the microcomputer industry. The objective is to help a member of the district management team decide what is needed and how to make the important decisions.

Step 1. - As you proceed with your quest for a microcomputer, do not become confused with what the equipment is called. The important matter is to find a machine that can do the job and also be cost effective.

Step 2. - Read all you can find concerning microcomputers, especially if you consider yourself a novice in this area. You should familiarize yourself with the basics by visiting with other organizations that have equipment, or purchase business or computer magazines which will provide you with much informative literature and advertisements.

Step 3. - Analyze your actual current needs for a microcomputer and outline your probable future needs. Forecasting the future workload for a microcomputer is sometimes difficult, but as one becomes familiar with the selected equipment, additional water management or office applications might appropriately be placed onto the computer system. Word processing and/or graphics capability are normal software enhancements that are beneficial. Additional memory, more advanced computer languages, more data storage capacity, and faster higher quality printers are some of the items you may desire in the future. The selected equipment should have the ability to expand as both your needs and your budget grow.

Step 4. - Now you should be ready to start looking at equipment that may be located in a local computer store. This will give you an opportunity to see what the microcomputer looks like, perhaps get some hands-on experience, and allow you to talk to sales people in a relaxed atmosphere. Also, you will have a chance to observe the type of functions this equipment can perform.

Step 5. - Examine the types of software needed to perform the various tasks you have outlined for the microcomputer. You'll most likely require a word processing package, a spreadsheet, and a data base manager. Beyond these, there are thousands of packages for business operations, such as project management, graphics, communications to other computers, and accounting. There is also software available to train personnel on how to use the microcomputer, and even teach how to type.

Step 6. - Spend some time talking to other businesses in your area that have been using microcomputer equipment similar to what you are interested in. Perhaps you can have discussions with management of other irrigation districts to learn what type of equipment they may already have or may recommend. Frequently, these conversations will reveal the problems of service or reliability encountered, and you will have an opportunity to review the types of applications placed on their microcomputer.

Step 7. - The financial picture now must be focused toward your board of directors and accountant. If you are anticipating purchase of a larger computer system, such as a mini or mainframe, the economics of lease or lease-purchase vs purchase might be considered. Also consider the depreciation and the investment tax credit available if the equipment is purchased. Approval to seriously shop for a computer system may have to be made at this time, if not already achieved through management channels.

Step 8. - Now you should be prepared to start shopping for your microcomputer, having a good idea of what applications will be placed upon it, what is available, and how much you can spend. The sales people should provide firm equipment configurations and related costs. The following list contains some of the technical items that should be discussed and included in the configuration.

a. Memory capacity. - The microcomputer must contain sufficient memory to satisfy the requirements of your current applications. A current memory standard of 640K is appropriate for most applications. More sophisticated software requires expanded memory between 640K and 1Mb, and some even require memory above 1Mb which is called extended memory. Make sure the equipment can have additional memory added if you expect it to perform more complicated and larger tasks in the future. The cost of future add-on memory should be considered.

b. Processor type. - There are various types of processors to consider. In the IBM/IBM-compatible market place, the 8088, 80286, and 80386 processors are common. The 8088 is the slowest, used in the XT machines. The 80286, also called the AT, is the current industry standard, soon to be replaced with the 80386, the fastest of the three. The 80486 processor will probably be the standard of the early 1990's.

c. Clock speed. - Various incremental speeds from 6-MHz to 33-MHz are commonplace. These numbers represent the internal clock crystal speed.

d. Disk storage. - Modern microcomputers contain both floppy and hard disk storage devices. Floppy disks, and the hardware drives that read and write to them, are available in two physical sizes: 5-1/4 inch and 3-1/2 inch. The 5-1/4 inch have two capacities, either 360Kb or 1.2Mb. The 3-1/2 inch also have two capacities, either 720Kb or 1.44Mb.

Hard disks begin at 10Mb, although 20Mb to 40Mb is the current starting range.

e. Programming. - The software availability is very important. Perhaps the vendor will provide packaged computer programs to accomplish your work, but often you will need additional programming expertise in your organization. Therefore, stay with the more basic programming languages such as BASIC and FORTRAN. Consider the cost of training personnel to program the microcomputer or the costs of packaged programs available from the vendor.

f. Printers. - The Dot-matrix printer is the common type that forms characters through a series of dots. The laser printer produces excellent results, is quieter, and performs at high speed. The Daisy wheel printer is similar to a typewriter, produces very good quality output, but is somewhat slow and noisy.

g. Maintenance. - This is a very important consideration to discuss with the vendor. Maintenance on a microcomputer can be through a cost-per-month arrangement, or some vendors of "microcomputer" equipment also offer carry-in servicing. Another procedure is to have knowledgeable personnel "fix-it-themselves," by replacing failed components.

Step 9. - The shopping is now finished so the final decision must be made regarding your microcomputer acquisition. A final meeting with the board of directors may be required. Again, justify the costs versus benefits to be expected from the equipment and be certain your organization is ready to make a commitment to use the equipment. Costs are currently decreasing in the microcomputer field as advancements are made in electronic technology, but a commitment today should improve your water management and office administration.

Step 10. - Make the commitment to acquire the equipment. Notify the district employees of the decision and collect the tools to help computerize your water management and office endeavors.

Glossary of Computer Terminology

(Capitalized words are defined elsewhere in the Glossary)

b (byte)

Used with K and M as in 360Kb (360K bytes) and 20Mb (20M bytes).

Backup

The process of making duplicate Disk/Diskette files on another disk/diskette or on cartridge tape. Backup also refers to the duplicate files. Backups are made to protect against Hardware malfunction or operator error.

BASIC

A high-level computer language Interpreter or Compiler, which is simple and easy to learn. There is a minimum set of industry-standard instructions which might be enhanced by each computer manufacturer or BASIC supplier. (BASIC = Beginner's All-purpose Symbolic Instruction Code).

Baud

Data transfer rate between computers or computer components in bits per second (bps). Usually, Baud/10 = characters per second (cps).

Binary Numbers

The binary system uses two symbols, 1 and 0, to perform all mathematical combinations and computations. The symbols are the presence or absence of a pulse of electricity within the computer. Numbers are represented as being so many ones, so many twos, so many fours, so many eights, etc., with each place doubling the prior place, going from right to left. The decimal number 8 would be written in binary as 1 0 0 0. Fifteen would be written 1 1 1 1. Binary numbers also represent alphabetical characters and special symbols.

Bit (binary digit)

One digit in a binary (base 2) number. All data and instructions are stored in a computer as binary numbers. Sometimes used to size computers. See Byte and Word.

Bus

A set of electrical lines that are used to connect one or more computer components together inside a computer's chassis.

Byte

One character or eight bits. Usually expressed in thousands (e.g., 360K bytes) or millions (e.g., 40M bytes).

Cassette

A magnetic tape cartridge. Often used on Microcomputers for disk backup or inexpensive Mass Storage.

Central Processor

Sometimes known as the CPU (central processor unit), it is the heart of the Microcomputer where the "thinking" is accomplished.

Chip

An Integrated Circuit module. It may be Primary Memory or it may be some part of a Microprocessor or I/O Device controller. Chips are usually plugged into a printed circuit board.

COBOL

A high-level computer language Compiler used for business applications. It is found on some Microcomputers and most larger computers. It is well standardized; it can be converted from one computer to another with few changes. (COBOL = COmmon Business Oriented Language).

Compiler

A Program that translates other programs from human-readable computer language instructions to machine-usable instructions. Examples: FORTRAN, COBOL, and some BASIC.

CP/M

A Microcomputer Operating System developed by Digital Research, not as popular since DOS was developed.

Desktop Computer

See Microcomputer.

DBMS (data management system)

A program package which allows the user to define file formats, input and update data, and create reports. It performs complex relationships and manipulates data quickly. More complex applications may require a computer professional or additional programming to set up and use effectively. A DBMS also has procedures to manipulate and integrate several files at once.

Disk

Often called "hard disk" to differentiate from a Diskette. A Mass Storage device which is thin, circular, and covered with a magnetic material. Microcomputers usually use the sealed, Winchester-technology disk.

Diskette

A flexible Mass Storage disk. Usually 5-1/4 inches square. The 3-1/2 inch version is a Mini Diskette. Called "floppy disk" and "mini floppy," respectively.

DOS

The Disk Operating System found on most IBM/IBM-compatible microcomputers.

Drive

A device used to turn or move Mass Storage media, allowing input from or output to the media.

File

A named block of text, data, program instructions, or commands. It may reside on Mass Storage but must be transferred to Primary Memory for manipulation and use. See Record.

FORTRAN

A high-level computer language Compiler used for engineering and scientific work. Standardization has allowed its use on all sizes of computers. FORTRAN Programs can be converted from one computer to another with little or no changes. (FORTRAN = FORmula TRANslator).

Hardware

The actual computer equipment without the operating system or programs required to make it function.

High-Level Language

See BASIC, COBOL, AND FORTRAN.

I/O (Input and Output) Devices

Electro-mechanical devices used to communicate with the computer user. Examples of input devices are typewriter-like keyboards, mouse(s), and digitizers. Output devices include monitor displays, printers, and plotters. NOTE: Mass Storage devices do provide input and accept output, and the process of communication with Mass Storage is commonly lumped into I/O operations. However, the phrases "input and outlet devices" or "I/O devices" do not usually include Mass Storage devices.

IC (Integrated Circuit)

A combination of electronic components that fit into a Chip, designed to serve a variety of purposes.

Instruction

The smallest part of a Program that a computer can execute.

Intelligent Terminal

Sometimes called a "smart" Terminal; it has limited capabilities for programming, text editing, and sometimes graphics. Plotters, printers, and other devices can be connected for additional capabilities.

Interface

1. The process of connecting two computer components, e.g., a Diskette drive to a computer processor.

2. The circuitry required to interface computer components.

3. Man-machine interface - the Hardware/Software or the process through which one uses a computer.

Interpreter

A Program that is like a Compiler except that it directly executes each instruction as it is translated. Example: BASIC.

K (Kilo)

Usually 10 raised to the 3rd power, or thousands. With computers it stands for, as an example, 640K bytes of memory or a 360K-byte diskette.

Large Computer

A computer which is best for applications requiring high speed, high accuracy, or large data volume. Example: CYBER 990. Typical System price: $1,000,000 plus. Application programs (including engineering) are abundant and professional, and are usually available on a Timesharing service. Memory and Peripheral expansion are very good, but very expensive. Usually sold by the manufacturer. Operations are by computer professionals only, but the user can be a non-computer professional (although he is often frustrated by a large computer's complexity). Almost always has many Terminals. Good Software transfer can be expected between Large Computers but only if in a compiled language. Super computers (such as Cray) are very fast and very expensive but not widely sold. Sometimes called a "Mainframe" computer.

Lease-Purchase

Rather than leasing the equipment for a flat charge per month or purchasing the equipment outright, an alternative some vendors provide is the lease-purchase agreement. Using this option, a percentage of the lease investment can be applied toward the purchase price at a later time.

Line Printer

A device which prints onto paper the output from the computer one line at a time. This is usually the more costly but faster type printer available for Mini and Large Computer installations.

M (Mega)

Usually 10 raised to the 6th power, or millions. With computers, it stands for, as an example, a 10Mb disk which contains around 10 million characters of information.

Mass Storage

A Hardware device for software and data storage, with access speeds in milliseconds. Normal storage is on Disk, Diskette, or tape media.

Matrix Printer

A device which prints onto paper the output from the computer one character at a time, with each character composed of a matrix of dots rather than solid lines.

Memory

That portion of the computer that contains the main storage, referenced as a number of words, such as 640K words. Thousands of transistors embedded in silicon constitute a memory Chip.

Microcomputer

Often called a "Personal Computer." A computer which is more powerful than programmable calculators, hand-held computers, and Intelligent Terminals, but less powerful, physically smaller, and less expensive than Mini and Large Computers. Most microcomputers are dedicated to a single user or to similar business or engineering functions. However, multiuser (multiterminal) capabilities are possible with more advanced Operating Systems such as OS/2 and UNIX.

Microprocessor

Often used when referring to Microcomputers, but it is only the "brain" of a Microcomputer. It manipulates data and outputs results according to programs. Microcomputers as well as larger computers may use microprocessors to control computer Peripherals such as printers or disk drives. The most popular microprocessors are the Intel 8088 (8-bit), 80286 (16-bit), and 80386 (32-bit).

Minicomputer

A scaled-down Large Computer. Example: Digital Equipment VAX 8300 systems. Typical System price: $30,000-$50,000. A mini is more powerful, physically larger, and more expensive than a Microcomputer. It usually requires computer-oriented operations personnel. Usually has several Peripheral devices. Uses all major computer languages. Application programs (including engineering) are abundant and usually professional. Memory and Peripheral expansion are very good but expensive. Usually sold by the manufacturer but can be purchased through third-party dealers. Operations are oriented to computer professionals, but the user is often a non-computer professional. Can support several Terminals. Good Software transfer between other minis and larger computers if in a Compiler language. Also see Super-Minicomputer.

Minidiskette

A 3-1/2 inch flexible disk, also called "mini floppy."

Modem

Hardware device used to communicate with Timesharing services or other computers, usually over telephone lines. Modems are either internal boards that plug directly into the computer circuitry, or are external devices that connect to the computer via a serial port. They normally communicate at speeds of 1200, 2400, or 9600 baud.

Monitor

A display similar to a TV screen composed of a matrix of dots which are excited by an electron gun to form characters or graphic images. For Microcomputers, monitors are available in Monochrome, CGA (Color Graphics Adapted), EGA (Enhanced Graphics Adapter), and VGA (Video Graphics Array).

Network

The process of connecting Microcomputers or Workstations together to share hardware and software resources. Often incorporates a centralized file server containing a large Disk for Mass Storage.

Operating System

Every computer contains a Software package known as the operating system. This handles the communications between the operator and computer, allows for data file management, and performs many other internal and external chores. Batch operating systems do not allow communications between the operator and computer once the job has begun. Interactive operating systems allow data to be entered as a job is executing. Real-time means the system responds to external demands on a priority basis, such as receiving telemetered data. Time-sharing operating systems allow multiple users to access the system and share its resources at the same time. Example of operating systems are DOS, OS/2, and UNIX.

PC (Personal Computer)

See Microcomputer.

Peripherals

An Input/Output or Mass Storage device which connects to the computer processor. Examples: Monitor, printer, Diskette Drive, and plotter.

Primary Memory

Built-in memory for program and data storage requiring immediate access by the computer. Access speeds are in microseconds. Also called RAM (Random Access Memory).

Program

A series of computer language instructions that exactly define a process, procedure, or calculation to be performed by a computer. Also called Software.

RAM (Random Access Memory)

Random Access Memory is a memory bank you can add or retrieve information at random. This is the area of the computer that will contain your program and the data used when the actual "thinking" process is underway. The contents of this memory are changeable.

Record

A part of a File containing related items which are called fields.

ROM (Read Only Memory)

Read only memory is memory that cannot be changed by the operator or programmer. It contains the essential instructions seldom changeable in day-to-day usage, such as the computer operation information.

Software

A loosely used term for computer Programs and anything not electro-mechanical (i.e., Hardware). The two main categories of software are "system" and "application." System software is generally provided by the manufacturer and makes the machine useful. Application software is the program generally written by or for the user to perform specific tasks. Third-party software is available in both categories.

Spreadsheet

A Program that manipulates text, numbers, and formulas in a matrix on a Monitor display. Useful for any set of financial or engineering calculations that can be visualized in two or three dimensions.

Storage Protection

The art of protecting programs and data located within the storage medium of the computer. Retaining Backup copies of all data and programs is feasible should a computer "crash" occur. Security of the data is also a prime consideration, especially for financial, payroll, and employee records.

Super-Minicomputer

A high-end Minicomputer similar to a Large Computer in every respect but price. Example: High end DEC VAX computers. Typical System price: $50,000 to $1,000,000. See Minicomputer.

System

A complete computer configuration including: a computer, system Software (Operating System), an Input Device (keyboard or mouse), an output device (printer), usually some Mass Storage (disk), and general-purpose application Software.

Terminal

A printing or video (Monitor) display and keyboard device which provides access to a Mini or Large Computer.

Text Editing

Changing files of text, program instructions, or data using editing commands. Not as automatic or specialized as Word Processing.

Timesharing

Simultaneous use of a computer by several people each with their own Terminal. Timesharing requires a specialized, multiuser Operating System.

UNIX

Popular multiuser Operating System from Bell Labs for Large Computers, Minicomputers, and Workstations. Also is being implemented in the Microcomputer environment. Complex and powerful, it is most effective for program development and graphics representation. Has been modified and enhanced by various vendors for their specific equipment.

Word

A unit of memory (Primary or Mass Storage). Measured in Bits. Varies by computer size: Microcomputers use 8, 16, or 32 bit words; Super-Minis, 32-48 bits; Large, 32-60 bits.

Word Processing

A Program that manipulates text by changing words, moving paragraphs, automatically paging and reformatting, and inserting names or paragraphs. It differs from text editing in that it uses more automatic features and keyboard control along with the manipulation of any line on the display.

Workstation

A highly sophisticated Desktop sized computer system with faster processing power than the Microcomputer that approaches or equals the Minicomputer. It is a multiuser and multitasking machine often using the UNIX Operation System, a window environment, and large Monitor allowing good graphical presentations.

Water Systems Operation and Maintenance Workshop - 1990

WATER SYSTEMS MANAGEMENT - PART 4

Session 2-4

Frank Dragoun and Rod Cox

1. Introduction

   a. Session 1-1 includes:

      (1) Relationship of manager and board as a team and their functioning together, respecting each other's rights and responsibilities
      (2) Principles of maintaining favorable relations among management, employees, and water users
      (3) Every water project is unique and its management requires a unique set of operating policies
      (4) Need for continual evaluation of practices, methods, procedures, and policies used in managing water supplies
      (5) Guiding rules for board, management, and employees

   b. Overall goal of this session - to promote discussion and exchange ideas, views, and experiences on project management and automation.

2. Responsibilities of Board of Directors

   a. The board of directors of a water district is responsible for the care, operation, and maintenance of the district.

   b. Each director should be willing to:

      (1) Attend all meetings and work with all directors and support board's final decisions
      (2) Take a definite stand and make decisions that are unbiased and in the best interests of the entire district
      (3) Maintain a progressive attitude and enthusiastic, cooperative approach toward attaining sound district improvements

   c. Establish and maintain up-to-date district policies in areas including:

      (1) Water allotments and delivery charges
      (2) Control of district property and expenditures
      (3) Employee wages and work conditions
      (4) Board-water user relations
      (5) District public relations

   d. Establish and approve budget and necessary funds

e. Hire and support competent manager

3. Responsibilities of Board President

a. Presiding officer at all board meetings.

b. A major responsibility of the board president is to make sure that actions taken by the board represent the considered judgment of the entire board.

4. Responsibilities of Manager

a. Under policies set by board, manager's responsibilities include:

(1) Keeping board informed on district activity
(2) Director of operation and maintenance of district facilities, equipment, and records
(3) Preparation and submission of an annual budget to the board
(4) Maintenance of an efficient work organization and employment and firing of personnel

b. In order for the manager to carry out policies established by the board, he must be delegated sufficient authority to implement the policies.

c. Communicate with board members and employees; make yourself "easy to talk to."

5. Board-Manager Relationship - Basic Rules

a. Manager may suggest policies and should be included in the establishment and evaluation of new policies or the changing of present policies; however, final acceptance and action are solely the responsibilities of the board.

b. Team effort idea is important. Decisions and agreement are made only by the board of directors and not by individual members acting independently; however, responsibility for putting the policies into effect should be clearly assigned to the manager.

c. It is especially important that the manager has authority and responsibility for hiring, firing, and directly or indirectly supervising all employees required for the operation and maintenance of the district under the personnel rules, regulations, and salary rates approved by the board.

d. It is an important basic principle that no member of the board attempts to direct the work of an employee, other than the manager, or interferes with active operation of the project except through the manager or at a board meeting.

6. Project Policy

a. Policy is the collection of rules and regulations under which the affairs of the project are to be managed.

b. A policy is not static. It must change with times and conditions as dictated by the needs of the water users. Sometimes the need for change is passed on through the manager to the directors. Often the manager must submit and recommend new procedures to the board. These should be discussed thoroughly by the board prior to being adopted. The more the board understands the need for new rules or procedures, the more likely the board members will feel that the policy is just and must be allowed by the project personnel. Change should not be made just for the sake of change.

c. The rules and procedures govern the duties of the board, manager, and other employees. They also define the manner in which water will be delivered and maintenance performed, and outline the privileges and responsibilities of the water users and the general public.

d. An important part of the policy is a well defined, written set of rules and regulations for water users and employees.

7. Selection of Personnel

a. This is another important phase of the project operation where a sound and definite board policy helps immeasurably.

b. By training and promotion of those who show the greatest ability and resourcefulness, a good organization can be built which can handle any emergency; that is, providing each employee is clearly given the authority to take the initiative in the routine applications of his or her particular job. Delegating responsibility without granting authority for individual initiative tends to produce an organization that sags and slows production when the supervisor is not available to issue instructions.

c. Selection of employees should be based on experience, ability, and fitness for the job. Most water project jobs require more than one skill. For instance, a ditchrider who can operate a tractor or drive a truck is much better than one who can only work a shovel. Hiring the best qualified person regardless of operating experience usually works out best. If a person is willing to learn, more than one job can usually be learned.

d. Selection of employees should also consider Federal laws regulating certain hiring aspects such as Fair Labor Standards Act and Equal Employment Opportunity Act (Effective Affirmative Action Plan).

8. Compensation and Fringe Benefits

a. The type of work done and the caliber of individuals who stay with you are often determined by the wages paid. If wages are lower than normal for the area, you cannot expect to keep good personnel. Perhaps the district needs to make a wage survey.

b. Also, if all employees are paid the same wage or receive pay increases regardless of the quality or quantity of work done on the job, they tend to lose interest in doing a better job. This should be done on a scheduled basis. This applies to staff and salaried employees as well. Periodic performance ratings should be made and provisions made for wage increases based on merit.

c. Cost of living adjustments are becoming increasingly common. Care should be exercised in setting up a standard cost of living adjustment method that under extreme inflationary times could not be granted.

d. Fringe benefits are becoming increasingly important and diversified. Common fringe benefits now include such items as vacation time off with pay, sick leave, jury duty, armed services reserve duty, funerals, etc. Other fringe benefits are health and life insurance and pension plans, in addition to (or substituted for) the Social Security program. One fringe benefit that is often helpful is an annual get together of employees and their spouses. Knowing each other better tends to lessen distrust of another.

e. Allowing employees to participate in workshops such as this is a fringe benefit which benefits the district more than the employee.

f. Districts which are under the OSHA (Occupational Safety and Health Act) already have a strong safety program, but those political subdivisions which are not covered by this act should have a strong safety program. A set of written safety regulations which are enforced and continually updated is needed.

9. Grievances

a. Complaints should be investigated immediately rather than let a sore spot fester within an organization.

b. When the grievance involves a matter of established policy, it is wise to schedule an employees' committee to meet with the board at its next regular meeting so that a better understanding can be reached.

10. Public Relations

a. The ditchriders, in their daily contacts with the water users, are important ambassadors of management. If they provoke or cause dissatisfaction as a result of what they say or of how they do their work, the manager will surely hear about it sooner or later. Problems of this type should be handled immediately.

b. A ditchrider or watermaster should treat each water user fairly, impartially, and courteously. This does not mean that all water users can be pleased, but when a ditchrider cannot satisfactorily answer a water user's complaint, the manager should be notified so he can discuss the complaint with the water user.

c. Requests for additional services or work by water users should be carefully investigated. Some of the requests, such as replacing open ditches with pipelines or concrete linings or straightening ditch alinements, may tend to be beneficial to the project. Other requests, such as additional turnouts, may be of more benefit to the water user than to the project. In determining the approval of such work, benefits to both the project and the water user should be considered.

d. Information to the press or radio should, wherever possible, be written out and a followup completed so as to insure that the papers and radio are factual.

e. Every effort should be made to maintain friendly relationships with the various kinds of recreationists having access to the project canals and reservoirs. This will tend to encourage cooperation from them in obeying safety rules and regulations. Close coordination and friendly relations are needed with your local and state game and parks commissions, game wardens, etc.

f. Accurate basic information concerning water supplies and timing of deliveries is essential in public releases. Quick, off-the-cuff answers on these and similar type requests for information can get project personnel in trouble. It is best to wait for releases of information until definite information can be provided.

11. District Service and Automation

a. Water managers and operational employees should evaluate their water management and handling programs and procedures to determine if automation and scheduling techniques can be used or improved in the general area of system automation.

b. Objectives of good water management: Consider and evaluate the district programs as follows:

(1) Purpose of the project
(2) Project operational policy
(3) Water supply forecasts
(4) Water measurement program
(5) Water records program
(6) Water quality standards
(7) Water rights

c. Administrative Services - Most administrative services in a district's operation can be done more efficiently if computer systems are used to do the many, but necessary, mundane tasks of recordkeeping as follows:

(1) Personnel
(2) Safety
(3) Equipment
(4) Salary computations
(5) Taxes
(6) Other

12. Public Service

a. Employees of the district should be encouraged to become a part of the community in which they live and, wherever feasible, serve local governmental bodies in some capacity.

b. Supporting and participating in state and national water organizations are helpful. This provides good public relations and also gives a means of preventing legislatures from enacting laws which would be extremely harmful, if not impossible, to operate under.

13. Summary Thoughts on How Good Relationship is Maintained Between Management and Employees

a. Mutual respect and being considerate of each other are important. Recognize individual talents of employees and fully consider employees' ideas.

b. Involve employees in action. Delegate authority. Do not hold on to too many details.

c. Give credit where it is due. Emphasize team aspects but recognize individual efforts. It is surprising how much can be done if you do not worry about who gets the credit.

d. Anticipate needs for new assignments. Stockpile work assignments so that work flows smoothly.

e. Help employees grow. Go to bat for employees.

f. Handle employee grievances promptly.

g. Keep standards consistent on how employees are evaluated but allow freedom of expression on how work is performed satisfactorily.

h. Stay current and competitive on compensation and fringe benefits.

14. Summary Thoughts on How Favorable Relationships are Maintained With Water Users and Public

a. Keep them informed and up to date. Provide opportunity for open house or tour of district facilities by water users and public. This can include periodic tours for water users and for others who may have some influence on future legislation or direct or indirect operation of your district.

b. Welcome calls and inquiries. Keep stressing with employees the importance of good public relations.

c. Remember you are working for the water users and for the good of the district and the area of influence. Make every effort to give each user equal treatment and attention.

15. Automation of Operations and Administrative Services

a. Automation of district's water facilities is a very important aspect of providing better and more efficient water service, control, and recordkeeping. It should be understood, however, that automation is expensive to install, maintain, and remodel. It should be done with great care and maintained with good, competent, professional help.

b. Computerization of administrative services with any of the numerous inexpensive systems now available should be considered a must for any district. Caution should be exercised, however, to prevent a natural tendency to overutilize a computer system.

Water Systems Operation and Maintenance Workshop - 1990

WATER MEASUREMENT

Session 3-1 - Frank Dimick
Session 3-2 - Russ Dodge and Cassie Klumpp

It is impossible to overstate the need for an improved water measuring program at this time, not only in a district, project, or region, but nationwide and even worldwide. Within the United States, critical examination will be made to determine quantity, quality, use, and waste of water. Plans will be formulated to extend the use of water. Rather than just finding and developing new sources, water can be less expensively provided by conservation and equitable distribution of district water. More extensive use of available water requires better measurement and management procedures.

Every cubic foot of water "saved" as a result of improving the measuring devices is more valuable than a similar amount obtained from a new source because the saved water can be produced at considerably less cost.

Accuracy in water measurement is important in the operation of any water distribution system. Even though a surplus of water may now exist on your project, it is time to begin accurate accounting of what happens to your water. Doing this will help you define and anticipate future needs of your district.

It is best to use standard devices since they are less expensive. Their general use generates more backup data and experience, making them potentially more reliable.

A truly standard device is one which has been fully described, accurately calibrated, correctly made or installed, and sufficiently maintained to fulfill the original requirements. Standard discharge tables or curves may then be relied upon to provide accurate water measurements. In maintaining a standard device, it is only necessary to check visually a few specified items or dimensions to be sure that the measuring device has not departed from the standard.

Many other devices are nonstandard because they have not been installed correctly and, therefore, do not produce standard discharges. Although these commonly used devices may appear to be standard devices, closer inspection often reveals that they are not and they must be calibrated to provide accurate measurements. Thus, proper inspection and care during installation and/or construction of standard water measurement devices is a characteristic of good management.

In rare instances, a nonstandard device is selected for a particular measurement situation, but more frequently a district finds itself unexpectedly stuck with a nonstandard device. This happens because a device has been installed improperly, is poorly maintained, is operated above or below the prescribed limits, or has poor approach (or getaway) flow conditions.

Accurate discharges from nonstandard structures can be obtained only from specially prepared curves or tables based on calibration tests, such as current meter ratings. Calibration tests can be quite costly when properly performed. Ratings must be made at fairly close discharge intervals over the complete operating range, and curves and/or tables prepared. It is, therefore, less costly and usually not too difficult to install standard devices and maintain them in good condition. Standard discharge tables may then be used with full confidence. In checking a nonstandard device, it is difficult to determine by visual inspection whether accuracy is being maintained except by recalibration.

In inspecting a water measuring station to judge or evaluate its probable accuracy, it should be determined whether the station uses a head or velocity measuring device. This is the key to the order and importance of other observations. In either case, the first observation should concern the visible flow conditions just upstream from the measuring device.

Measuring devices themselves may be at fault in producing measurement errors rather than the flow conditions. The faults may be divided into two types - those caused by normal wear and tear, and those resulting from poor installation.

It is possible to obtain inaccurate discharge measurements from regularly maintained equipment properly installed in an ideal location, if poor measuring techniques are used by the operator. Measurement of head is very important and some of the techniques now in use are not compatible with the relationships between head and discharge known to exist.

Approach Flow

Extremely large errors in discharge indication can occur because of poor flow conditions just upstream from the measuring device. In general, the approaching flow should be the same as tranquil flow in long, straight canals (without obstructions) of the same size. Any deviation from a normal horizontal or vertical flow distribution, or the presence of water surface boils, eddies, or local, fast currents, is reason to suspect the accuracy of the measuring device.

Sand, gravel, or sediment bars submerged in the approach channel; or weeds or riprap obstructions along the banks or in the flow area can cause unsymmetrical approach flow. Other causes may be too little distance downstream from a drop, check, turnout, or other source of high-velocity or concentrated flow; a bend or angle in the channel just upstream from the measuring device; a too rapid expansion in the flow section; or an eddy tending to concentrate the flow in part of the cross section.

Some devices have been standardized with certain or minimum velocity of approach conditions that must be provided and maintained.

Turbulence

Turbulence is the result of relatively small volumes of water spinning in a random pattern within the flow mass as it moves downstream. It may be recognized as water surface boils or eddies which appear and disappear in a haphazard way. Because of this local motion within the general motion of the flow mass, any particle of water may at any given instant be moving forward, sideways, vertically, and even backward. In effect, then, the water is passing a given point with a start and stop motion rather than with a uniform velocity which is ideal.

Excessive turbulence will adversely affect the accuracy of any measuring device, but is particularly objectionable when using current meters or propeller meters of any kind. Turbulence can be objectionable even without the "white water" caused by air entrainment, which is often associated with it. Turbulence can be caused by a stilling basin or other energy dissipator immediately upstream, by a sudden drop in water surface, or by obstructions in the flow area such as operating or nonoperating turnouts having projections into or indentations from the flow. Shallow flow passing over a rough or steep bottom can also be the cause. Weeds or riprap slumped into the flow area or along the banks, or sediment deposits upstream from the measuring device, also can cause excessive turbulence.

Rough Water Surface

A rough water surface, other than wind-generated waves, can usually be eliminated by reducing turbulence or improving the distribution of the approach flow. A rough water surface can cause errors in discharge measurements when it is necessary to read a staff gage to determine head or determine the cross-sectional area of the flow.

Poor Workmanship

Contrasting with the measurement devices which were once accurate and dependable, and have deteriorated, are those which because of poor workmanship were never a standard device. These include devices which are installed out of level or out of plumb, those which are skewed or out of alinement, those which have leaking bulkheads with flow passing beneath or around them, and those which have been set too low or too high for the existing flow conditions. Inaccurate dimensions can also be the cause of measurement error.

Pressure readings are needed to determine discharges through certain types of meters. Piezometers, or pressure taps as they are sometimes called, must be regarded with suspicion when considering accuracy of flow measurements. Piezometers must be properly installed with care and with a knowledge of how they perform, otherwise the pressure values they indicate can be in error. They should be normal to the flow and free from burrs and projections into the flow.

Weathered and Worn Equipment

An unwelcome but fairly common sight on older irrigation systems are weir blades, which were once smooth and sharp, in a sad state of disrepair. Edges are dull and dented. The blade is pitted with large rust tubercles. Weir plates are discontinuous with the bulkheads and are not vertical. Weir blades have sagged and are no longer level. Staff gages are worn and difficult to read. Stilling well intakes are buried in sediment or partly blocked by weeds or debris. Parshall flumes are frost heaved and out of level. Meter gates are partly clogged with sand or debris and the gate leaves are cracked and warped.

These and other forms of deterioration are often the causes of serious errors in discharge measurements. This type of deficiency is difficult to detect because normal wear and tear may occur for years before it is apparent to a person who sees the equipment frequently. However, it is readily apparent to an observer viewing the installation for the first time.

Measuring devices which are rundown are no longer standard measuring devices, and indicated discharges may be considerably in error. To be certain of the true discharge, the devices should be rehabilitated and/or calibrated.

Of necessity, proper management requires keeping up with maintenance. However, preventive maintenance is better in terms of public relations and employee morale, and on a long-term basis is less costly than reactive maintenance done after breakdown or under the pressure of customer complaint.

Practically all measuring devices, when in rundown condition or when improperly installed, deliver more water than they indicate. The very nature of most measuring devices makes it impossible for a device to deliver less water than it indicates. Therefore, water accounting records may not show a proper division between water used and water lost through seepage or waste. Proper evaluation of losses is necessary to establish the economic advisability of providing canal linings. Canal linings obviously cannot help to recover water lost through poor measuring equipment or procedures.

Exit Flow Conditions

Exit flow conditions can cause as much flow measurement error as some of the approach flow problems. However, in practice these conditions are seldom encountered. In general, it is sufficient to be sure that backwater does not or tend to submerge a device designed for free flow.

Faulty Head Measurement

Improper gage location or an error in head measurement can result in very large discharge errors. Readings obtained from stilling wells, whether they are visual or recorded, should be questioned unless the operator is certain that the well intake is not partially or fully clogged. Data from an overactive stilling well can also be misleading, particularly if long-period surges are occurring in the head pool. In fact, all head determinations should be checked to be sure that the instantaneous reading is not part of a long-period surge. Sufficient readings, say 10, should be taken at regular time intervals, say 15 seconds, to obtain the average head. More readings may be required if it is apparent the pool is continuing to rise or fall. If this is too time consuming, the cause of the instability should be removed.

Readings from gages or staffs which may have slipped or heaved should be avoided. Periodic rough checks can sometimes be made with a carpenter's level or square from a reference point on another structure.

Infrequent Measurement

When a head or velocity measurement is made to determine discharge, it can be concluded that the measured discharge occurred only at the moment of the measurement. It cannot be concluded that the discharge was the same even 5 minutes later or 5 minutes earlier. Therefore, water deliveries can be accurate only if enough measurements are made to establish the fact that the discharge did or did not vary over the period of time that water was being delivered.

In many systems, measurements are made only once a day or only when some mechanical change in supply or delivery has been made. Problems introduced by falling head, rising backwater, gate creep, or hunting are often ignored when computing a water delivery.

Sometimes when the discharge in the supply system remains constant, the water level or velocity reading changes because of a change in control or because checks have been placed in operation. Temporary changes in discharge in the main supply system may occur, for example, because water in effect is being placed in storage as a result of the rising water level. Changing water level may make it necessary to take more frequent head readings.

Use of Wrong Measuring Device

Every water measuring device has limitations of one kind or another and it is impossible to choose one device that can be used in all locations under all possible conditions. It is to be expected, therefore, that for a given set of conditions there may be several devices which would be suitable, but none could be considered entirely satisfactory. If flow conditions change or are changed by modified operations, an original device which was marginal in suitability may be found to be totally inadequate. It is possible, too, that the wrong device was selected in the first place. Whatever the reason, there are instances where accurate measurements are being attempted using a device which cannot, even with the greatest care, give the desired results.

Water Systems Operation and Maintenance Workshop - 1990

WATER RELATED SEDIMENT PROBLEMS

Session 4-1

Robert I. Strand

Introduction

The types of sediment problems encountered either at a diversion structure or throughout a water distribution system may vary from location to location, but quite often have many comparable characteristics. The variations are usually attributable to differences in river channels and river hydraulics, sizes of sediment from silts to gravels and cobbles, topographic changes, geology, and soil types. Despite the differences encountered, there are many similarities that permit applying the solution of a problem in one area to correct problems in other areas. Operation and maintenance problems involving sediment to be discussed are: (1) diversion structures and canal headworks, (2) canal and distribution systems, (3) pumping plants, (4) stability of natural channels to convey waste flows and return flow, and (5) miscellaneous river channel or sediment problems.

Sediment problems most often encountered under the five classifications are:

1. Diversion Structures and Canal Headworks

   a. Deposition of sediment in river channel above a diversion dam and downstream bed and bank scour.

   b. Sluicing arrangement not effective in reducing sand inflow to canal.

2. Canal and Distribution Systems

   a. Cleaning sediment deposits from canal.

   b. Need for settling basin at upper end of canal.

   c. Canal siphons under river channels not buried deep enough for high flood peaks.

   d. Cross drainage culverts sized for flood peaks as well as sediment loads.

   e. Seepage from canal with clearer water diversion.

3. Pumping Plants

   a. Sediment deposits in intake channels to plant.

b. Wear factor for pumping sediment-laden water.

4. Stability of Natural Channels to Convey Waste Flows and Return Flow

a. Excessive scour in wasteway channels.

b. Return flows may also upset stability of natural channel, resulting in bed and bank erosion.

5. Miscellaneous River Channel or Sediment Problems

a. Unstable river channels when used as conveyance channels.

b. Natural bank erosion may threaten farmland and water distribution system.

c. Techniques for stabilizing a river channel.

d. Erosion action of sediment in water.

e. Marina entrances.

Sediment and River Channel Characteristics

Before delving into a detailed description of the various operation and maintenance problems involving sediment, some basic concepts are important for a better understanding of the cause of these problems. A knowledge of these concepts will help in arriving at the solution or control of many of these problems. In describing some of these basic relationships, a description will be given of the characteristics relative to sediment and river channel hydraulics, omitting the detailed theory involved.

Sediment is primarily transported by water in rivers either as suspended load or bedload. The bedload is defined as the sediment moved along in essentially continuous contact with the streambed. The size of sediments being transported is very important and is classified by size variation into clay, silt, sand, gravel, or cobbles. Figure 1 shows a typical size analysis of the suspended as well as bed material sizes. This distribution is typical of many rivers in the Western United States, with exception of the higher mountain streams which may have gravel and cobble material in the bed and only clay and silt sizes in suspension. The sampler used to collect a suspended sediment sample for laboratory analysis of concentration and size distribution is shown in figure 2. Another important sediment relationship is the variation in concentrations of sediment within the depth of a river channel. Figure 3 shows the variation in the vertical for various size classifications. It should be noted that for the coarser size material in the sand size range that concentrations at the bottom of the suspended sediment area increase considerably and, if this increase is extrapolated down to the streambed, extremely high concentrations are encountered. The variations shown in

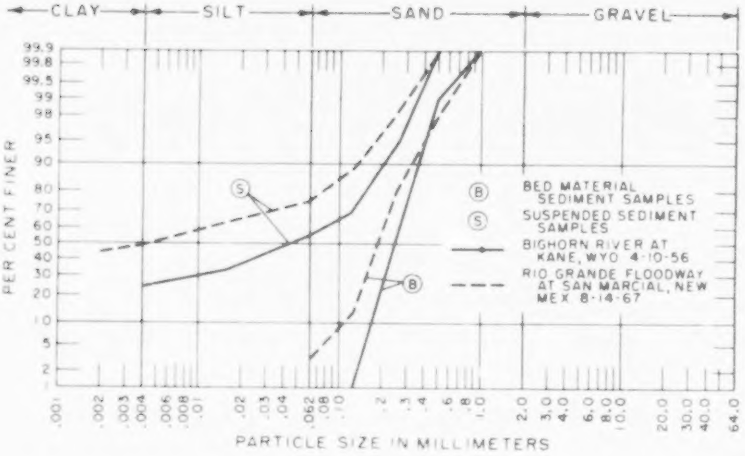


Figure 1. - Sediment size analysis.

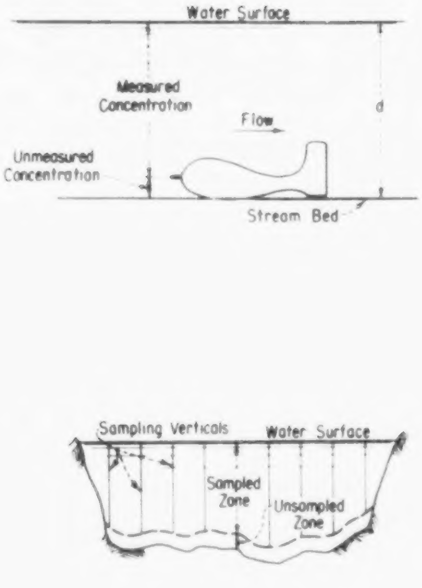

Water Surface
Measured Concentration
Flow
d
Unmeasured Concentration
Stream Bed
Sampling Verticals
Water Surface
Sampled Zone
Unsampled Zone

Figure 2. - Measured and unmeasured load. Suspended load sampling.

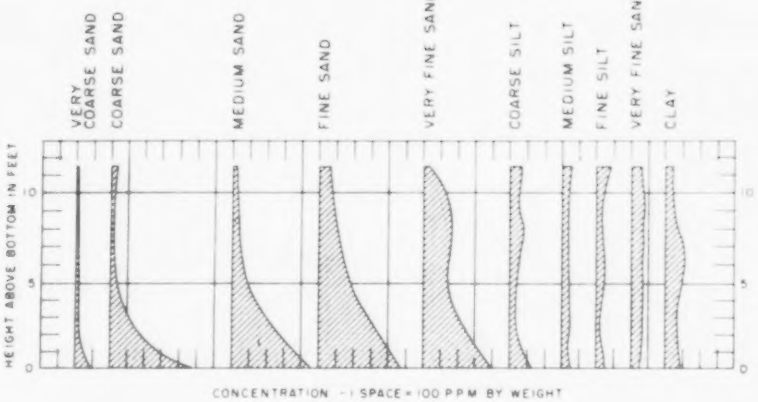


Figure 3. - Variation of sediment concentration in the vertical by size range.

figure 3 result from a combination of the velocity of riverflow with the fall velocity of the sediment particles. As can be expected, the larger size sand falls much faster and, therefore, has much lower concentrations near the water surface. There are many equations and theoretical techniques for computing the transport of sediment. Principally, any increase in river velocity causes an increase in size of sediment that can be moved. A general relationship of velocity versus size of sediment is shown in figure 4. This shows at what velocity sediment particles will either be scoured, transported, or deposited.

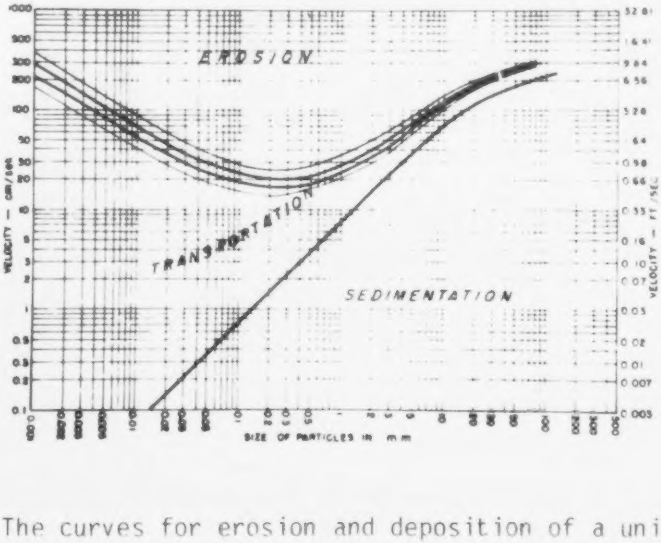


Figure 4. - The curves for erosion and deposition of a uniform material.

There are some general river channel characteristics directly related to sediment transport that help in analyzing many problems. A classic illustration within the Bureau of Reclamation is the channel stability balance. This balance, showing variations in sediment and in water (fig. 5), illustrates a logical explanation for unstable conditions with changes in either the sediment or water.

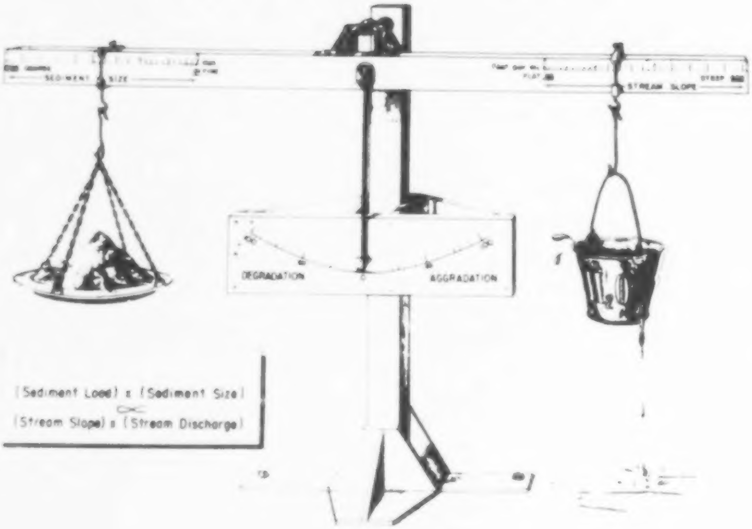

SEDIMENT SIZE
FLAT
STREAM SLOPE
STEEP
DEGRADATION
0
AGGRADATION
(Sediment Load) x (Sediment Size)
∝
(Stream Slope) x (Stream Discharge)

Figure 5. - Stable channel balance.

River channel hydraulics variations caused by flow conditions, as well as sediment movement, can be illustrated by the meander pattern of a river. Figure 6 shows a typical meander with sandbars on the inside bend or convex curve and deeper scour holes on the concave or outside bends. An example of the type of cross section of river that develops on the bends

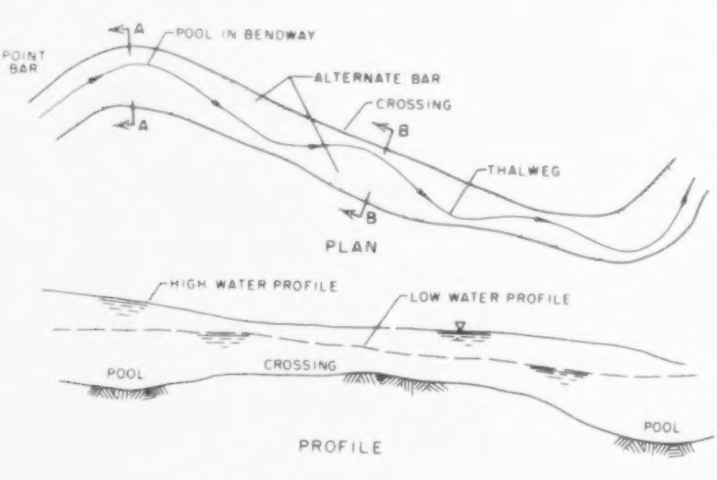

A
POOL IN BENDWAY
POINT BAR
ALTERNATE BAR
CROSSING
A
B
THALWEG
B
PLAN
HIGH WATER PROFILE
LOW WATER PROFILE
POOL
CROSSING
POOL
PROFILE

Figure 6. - River meander patterns.

and crossover points (fig. 7) provides some understanding of the movement of sediment. Secondary currents acting within the water prism form this channel configuration. The secondary currents are hard to visualize, but they are currents operating normal to the river velocity. As shown in figure 7, the secondary current pulls or draws any sediment movement in the river bottom toward the inside of a bend and deposits the sediment. Photographs illustrate this characteristic of sandbar movement at river bends for Frenchman Creek in Nebraska, Lake Fork in Colorado, and along the lower Colorado River between Arizona and California (figs. 8, 9, 10, and 11).

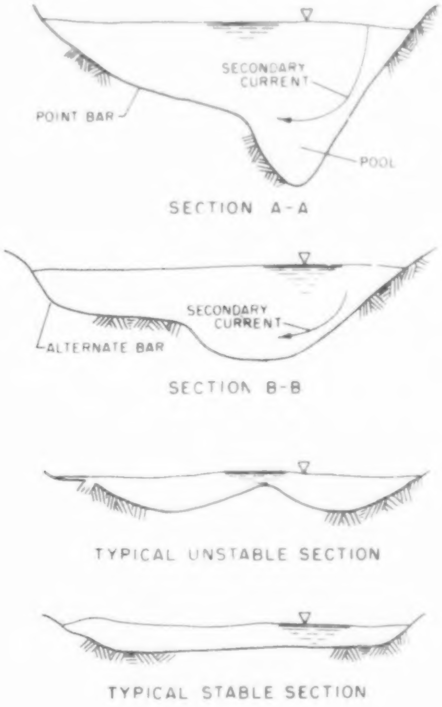

SECONDARY CURRENT
POINT BAR
POOL
SECTION A-A
SECONDARY CURRENT
ALTERNATE BAR
SECTION B-B
TYPICAL UNSTABLE SECTION
TYPICAL STABLE SECTION

Figure 7. - River cross sections.

Diversion Structures and Canal Headworks

At diversion structures, the volume of upstream sediment deposition varies considerably. We have found upstream aggradation or deposition even with sluiceways with invert near streambed elevation that are designed to transport the coarser materials into the downstream river. This results primarily from the large amounts of sediment being transported during high flood periods when the height of the main dam structure will create a higher upstream profile. An example profile is that

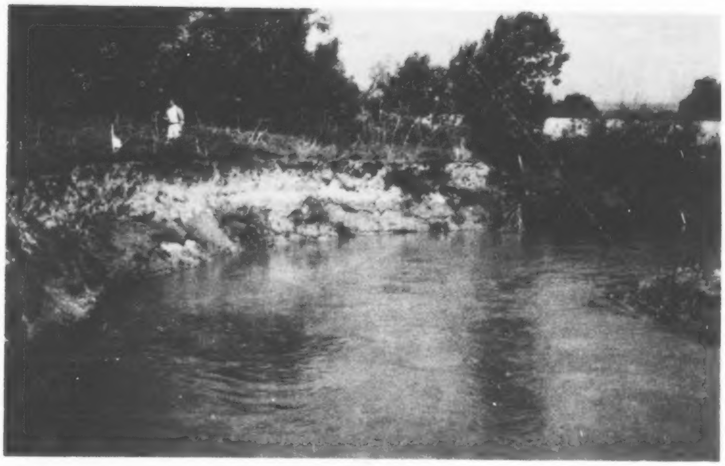


Figure 8. - Looking upstream on Frenchman Creek near Wauneta, Nebraska.

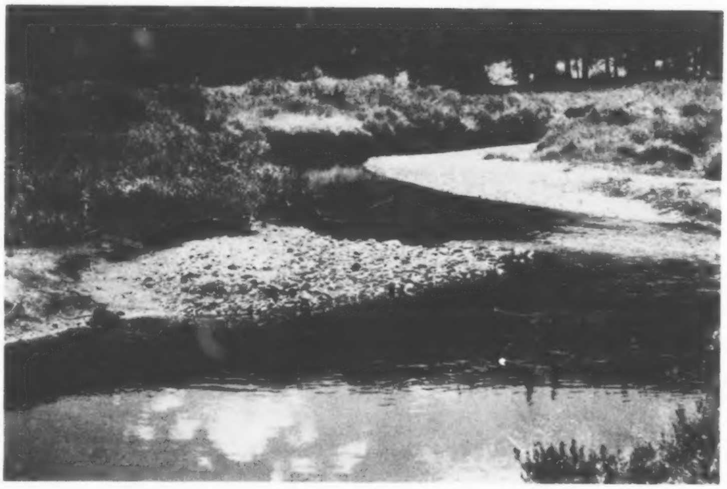


Figure 9. - Lake Fork channel near Leadville, Colorado.

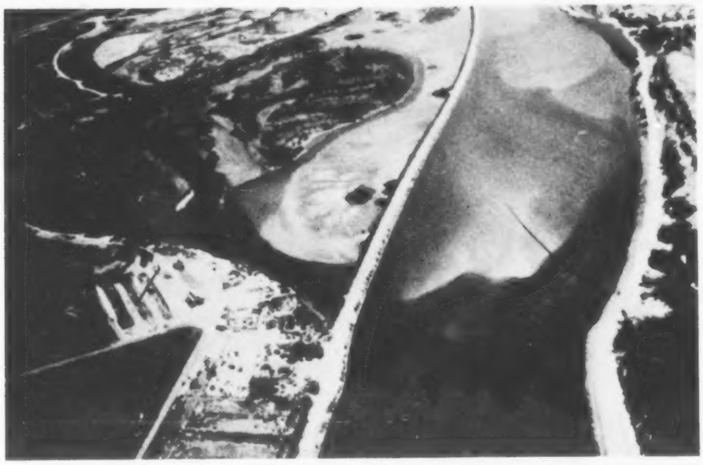


Figure 10. - Colorado River near Blythe, California.

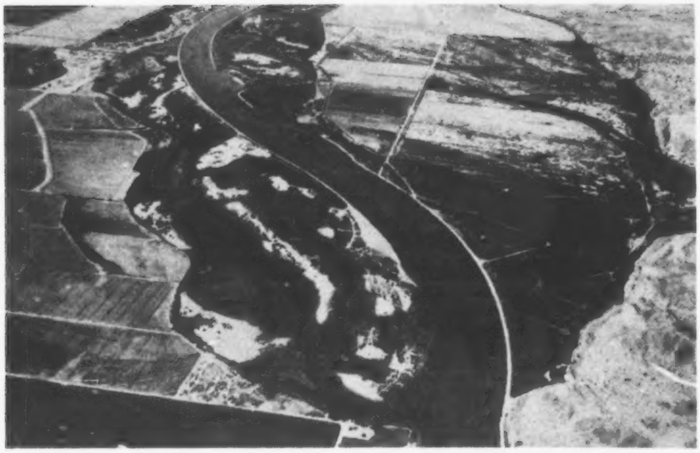


Figure 11. - Colorado River near Blythe, California.

for the Milburn Diversion Dam on the Middle Loup River shown in figure 12. The upstream extent of this aggradation depends upon the sediment transporting characteristics of the river and flow conditions. In these cases, it is advantageous to keep the sluice gate open during the flood and nonirrigation season. Many times the narrow width of a sluiceway will develop a low flow channel streambed elevation near the original streambed profile. Associated with deposition above a dam, quite often degradation below the dam is found. This is directly related to the sediment balance whereby the clearer water through or over the dam will cause some vertical scouring in the downstream river. In addition to vertical scouring, the clear water will also scour the riverbanks. Upstream aggradation and some downstream degradation have occurred at Milburn Diversion Dam, as shown in aerial photographs of the area (figs. 13 and 14).

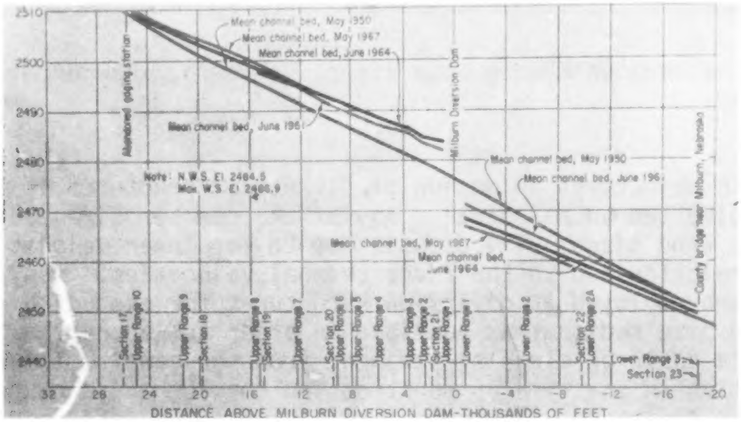


Figure 12. - Profile of the Middle Loup River.

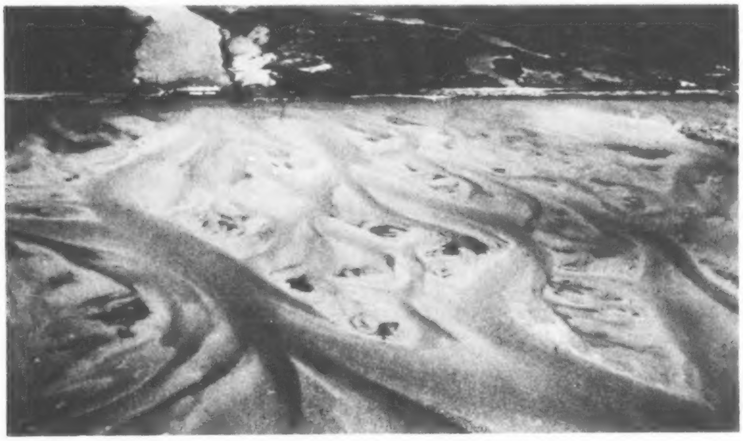


Figure 13. - Downstream view on Middle Loup River, Nebraska, showing Milburn Diversion Dam and upstream sediment deposits.

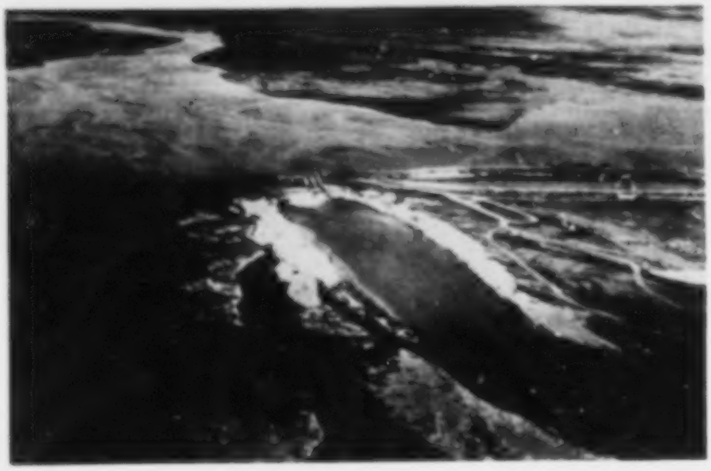


Figure 14. - Upstream on Middle Loup River, Nebraska, showing Milburn Diversion Dam.

One of the main objectives in design of diversion structures is to reduce the sediment diverted into a canal. Any of the coarser size sediment, especially the sand sizes, will deposit due to the lower velocities of the canal in relationship to the river channel velocities. Many techniques have been employed at diversion structures for excluding as much of this coarse size sediment as possible in order to reduce the operation and maintenance costs of cleaning sediment from the canal. There has been an evolution in the type of construction considered adequate for excluding these sediments. Throughout this country we find many diversion structures built prior to the 1940's that employed a principle of skimming off the top water having lower sediment concentrations, thereby reducing the inflow of sediment to the canal. An example of this skimming technique is shown in figure 15. Photographs of the skimming principle are shown at the bifurcation on Albuquerque Main Canal below Angostura Diversion Dam on the Rio Grande in New Mexico (figs. 16 and 17). These show conditions during diversions and subsequent to diversions and exemplify the problems of the skimming technique when diversions are made at an angle to the normal flow. Turbulence is developed sufficiently to suck much of the sand being transported, both in suspension and along bed, into the canal. Although many skimming devices have worked better, there is still a problem, under a range in flow conditions, when the velocity required to divert water off at an angle and into the canal creates a turbulent condition that defeats the principle of skimming.

With the advent of model testing and since the 1950's, new techniques were found for excluding sediment from a canal. The designs now used by the Bureau involve that of combining the theory of secondary currents that occur on bends of a river with the changes of sand concentrations in the vertical. An example of this is shown in figure 18 where a curved guide wall creates a bend to the flow. An illustration of this is the Courtland headworks on the Republican River (figs. 19 and 20). The

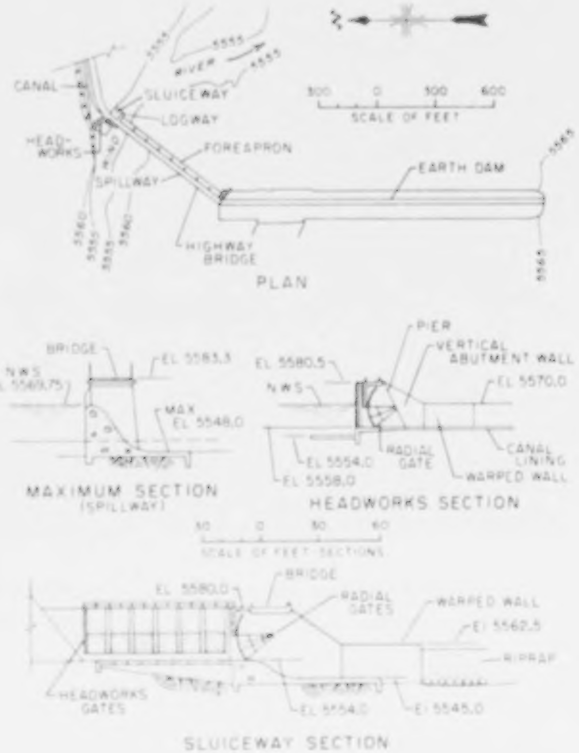

N
CANAL
RIVER
5555
5555
5555
SLUICEWAY
LOGWAY
HEAD-WORKS
FOREAPRON
SPILLWAY
EARTH DAM
300 0 300 600
SCALE OF FEET
5565
5560
5555
5555
5560
HIGHWAY BRIDGE
5565
PLAN
BRIDGE
EL 5583.3
N.W.S. EL 5569.75
MAX EL 5548.0
MAXIMUM SECTION (SPILLWAY)
PIER
VERTICAL ABUTMENT WALL
EL 5580.5
N.W.S.
EL 5570.0
RADIAL GATE
CANAL LINING
EL 5554.0
EL 5558.0
WARPED WALL
HEADWORKS SECTION
30 0 30 60
SCALE OF FEET-SECTIONS
BRIDGE
EL 5580.0
RADIAL GATES
WARPED WALL
El 5562.5
RIPRAP
HEADWORKS GATES
EL 5554.0
El 5545.0
SLUICEWAY SECTION

Figure 15. - General plan and sections of skimmer.

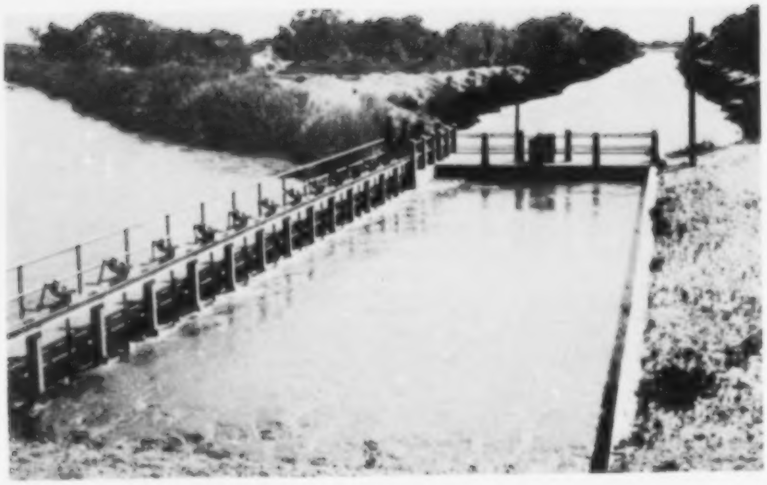


Figure 16. - Bifurcation on Albuquerque Main Canal below Angostura Diversion Dam on Rio Grande near Bernalillo, New Mexico.

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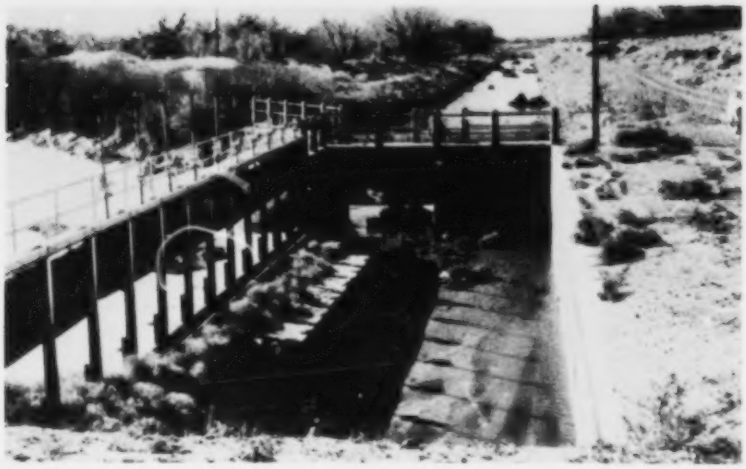


Figure 17. - Bifurcation on Albuquerque Main Canal below Angostura Diversion Dam on Rio Grande near Bernalillo, New Mexico.

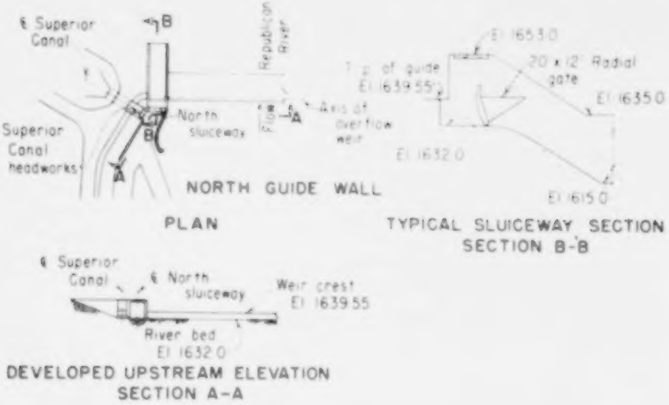

℄ Superior Canal
B
Republican River
North sluiceway
Flow
A
Axis of overflow weir
Superior Canal headworks
B
A
NORTH GUIDE WALL
PLAN
El 1653.0
Top of guide El 1639.55
20 x 12 Radial gate
El 1635.0
El 1632.0
El 1615.0
TYPICAL SLUICEWAY SECTION
SECTION B-B
℄ Superior Canal
℄ North sluiceway
Weir crest El 1639.55
River bed El 1632.0
DEVELOPED UPSTREAM ELEVATION
SECTION A-A

Figure 18. - General plan and sections of guide wall.

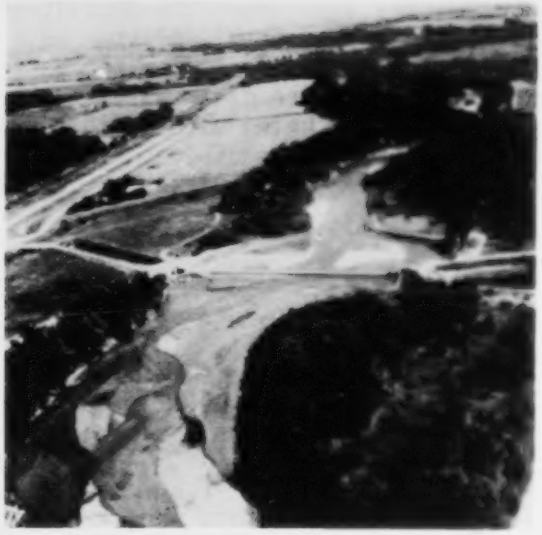


Figure 19. - Superior-Courtland Diversion Dam on Republican River near Superior, Nebraska.

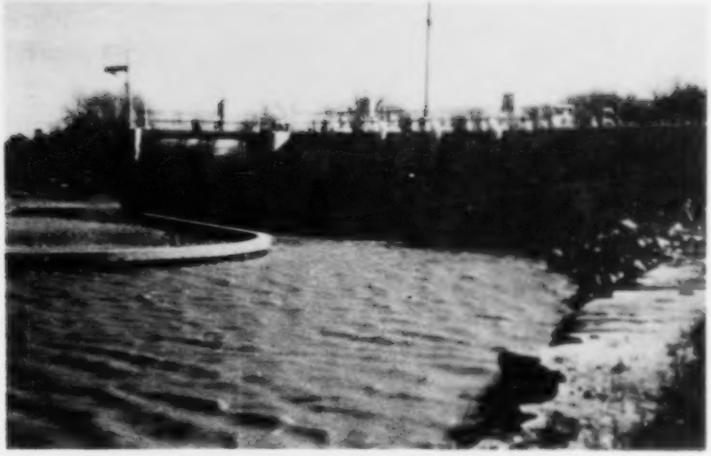
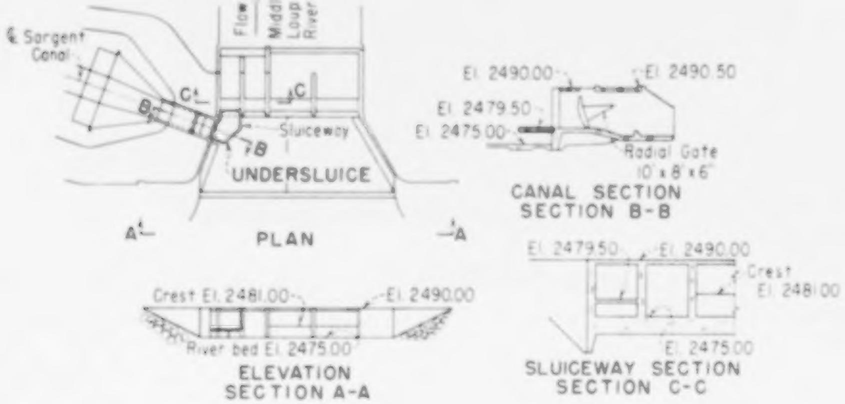


Figure 20. - Courtland headworks on Republican River near Superior, Nebraska.

aerial view of this headworks also shows the deposition upstream from the dam. Shown in figure 21 is a horizontal splitter weir that splits the vertical and sluices the coarse sediment near the bottom into the downstream river channel and diverts the clear water on the top into the canal. This modification is now being used by the Bureau on many sandbedded streams. Figure 22 is a photograph taken during construction of Milburn Diversion Dam showing this splitter weir principle.



℄ Sargent Canal
Flow
Middle Loup River
C℄
C
B
Sluiceway
B
UNDERSLUICE
A℄
PLAN
A
El. 2490.00
El. 2490.50
El. 2479.50
El. 2475.00
Radial Gate 10' x 8' x 6"
CANAL SECTION
SECTION B-B
El. 2479.50
El. 2490.00
Crest El. 2481.00
El. 2475.00
SLUICEWAY SECTION
SECTION C-C
Crest El. 2481.00
El. 2490.00
River bed El. 2475.00
ELEVATION
SECTION A-A

Figure 21. - General plan, elevation, and sections of undersluice.

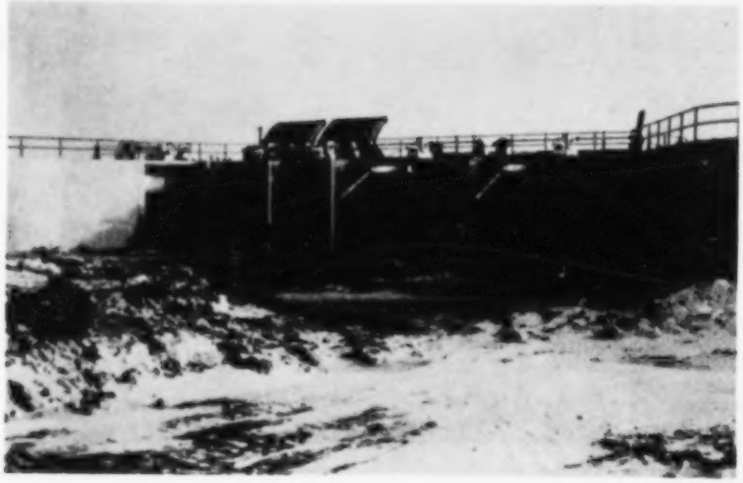
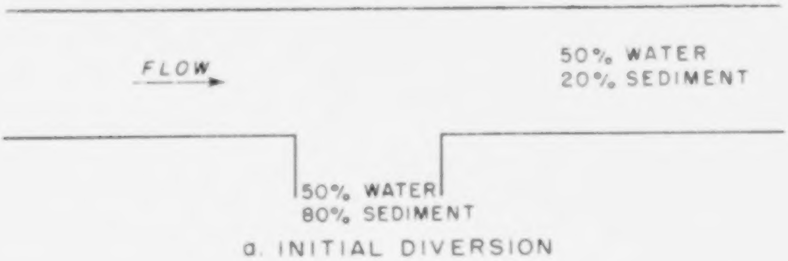
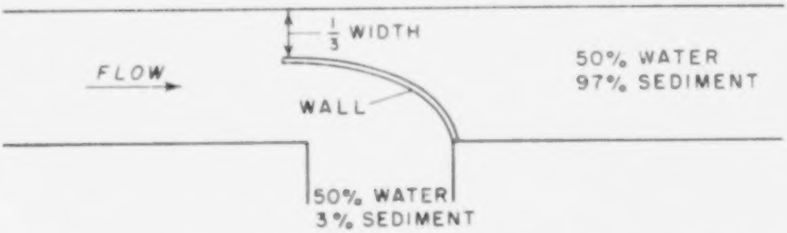


Figure 22. - Milburn Diversion Dam on Middle Loup River in Nebraska.

To illustrate some of the problems of diverting water into a canal, an example used by Dr. H. A. Einstein, formerly at the University of California, for students beginning their work in sedimentation is quite beneficial. Figure 23a shows a straight reach in which 50 percent of the water will be diverted at a 90° angle from the main channel flow. Under these conditions of sand being transported as bedload, the student first discovers that, due to the hydraulics of diverting flows at right angles, approximately 80 percent of the sediment will be diverted with 20 percent passing on down the channel. The objective of this problem is to design a device that will reverse this trend and show a smaller amount of sediment being diverted. Shown in figure 23b is the scheme quite often arrived at whereby the amount of sediment diverted is reduced to as low as 3 percent of the total by adding a curved wall.



a. INITIAL DIVERSION



b. WALL INSTALLED TO IMPROVE DIVERSION

Figure 23. - Sketch of 90° diversion of water and sediment.

In a recent discussion with Mr. H. I. Schwartz from the University of Witwatersrand at Johannesburg in South Africa, he described a somewhat different excluder principle being used on many South African streams. A summary of Mr. Schwartz's basic ideas regarding sediment at diversions involves three criteria to decrease the sediment being diverted: (1) locate the diversion point at a concave bend; (2) under an existing diversion, reduce the sediment by creating a bend by use of groins or solid boundaries; and (3) aline the diversion facing upstream with the axis parallel to the surface flow lines. To increase the sediment diverted, the diversion should be alined just downstream of a convex bend or inside of a bend and should be normal or 90° to the flow line.

It is interesting to note that in the correction found by the students in sedimentation at the University of California working under Dr. Einstein, the design technique for reducing the sediment diverted involves alining the diversion to parallel the direction of the riverflow. An interesting corollary to this technique of alining the diversion to parallel the surface flow is that of a bridge pier located in the middle of a river. Observations at bridge piers show the surface flow to plunge at the upstream face of the pier and create scour holes around the pier. This plunging action thus moves all sediment away from the pier and to both sides. An illustration using this principle at a diversion is shown in figure 24. The island in figure 24 was constructed upstream in order to force the riverflow to be alined parallel to the diversion. The top water of lower sediment concentrations is then diverted into the canal and the bottom sediments are moved along the bed and away from the headworks to a downstream sluice.

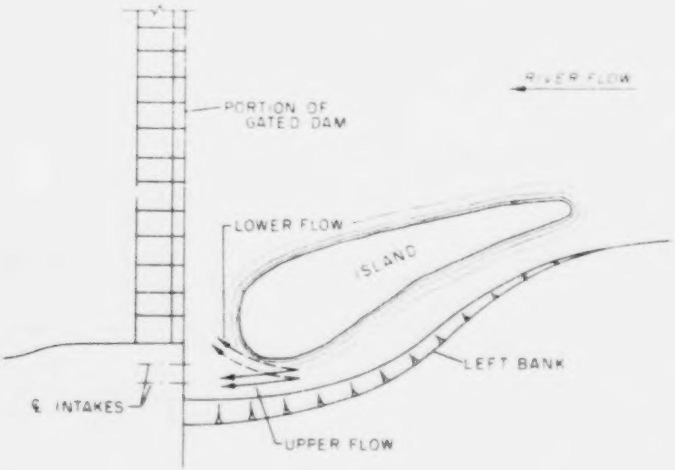


Figure 24. - Plan of diversion alined parallel to riverflow.

Canal and Distribution Systems

At many diversion structures where sluicing water is not available and/or the river carries sand-size material in suspension, the diversion of sand cannot be avoided. The flatter slopes in a canal and lower velocities are not capable of transporting the coarser sand size material. The result is that sediment is deposited in the canal as shown in figure 25.

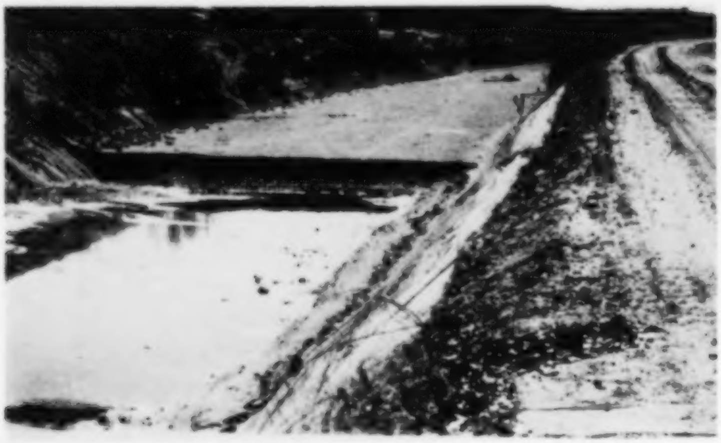


Figure 25. - Accumulated sand in South Canal, Salt River Project, Arizona.

Canal deposits are cleaned by dragline as shown in figure 26. A settling basin similar to that shown in figure 27 is often constructed in the upper reach of a canal to reduce the cost of cleaning sediments from a canal.

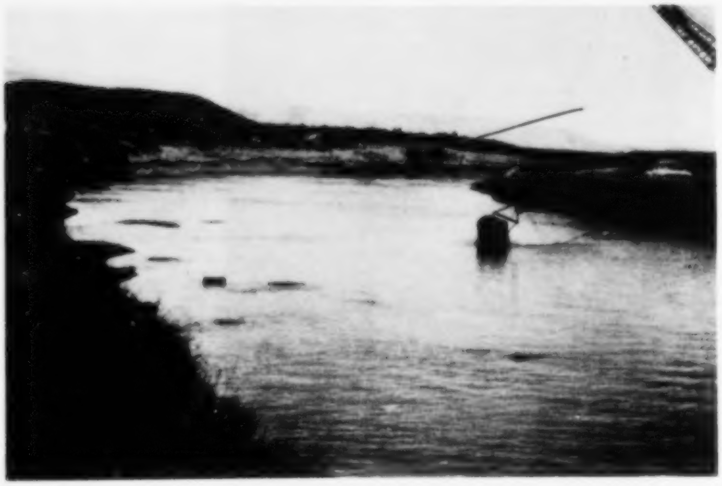


Figure 26. - Cleaning Wyoming Canal below Wind River Diversion Dam, Riverton Unit.

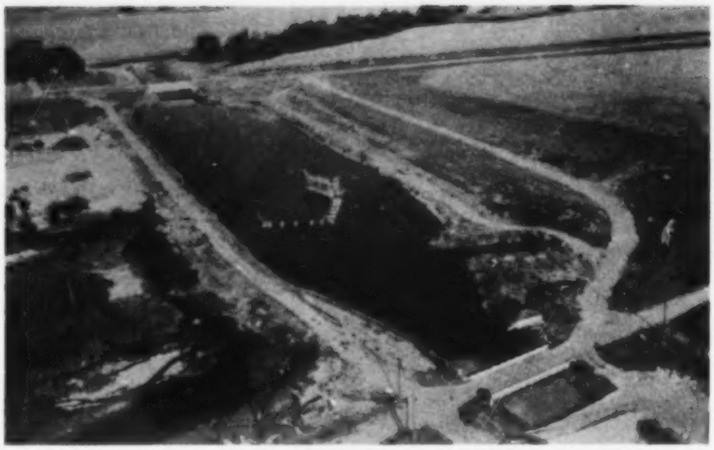


Figure 27. - Culbertson settling basin on Frenchman Creek, Nebraska.

The Bureau has constructed conveyance channels with slopes approximating those of the river. On these conveyance channels the design velocities are much higher than those of flatter slope canals and, therefore, some means of providing bank protection may be involved in the design. An example of this is the bank erosion along the conveyance channel on the Middle Rio Grande Project above Elephant Butte Reservoir, figure 28.

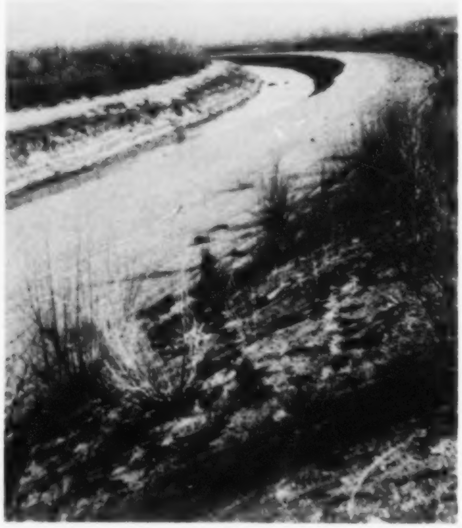


Figure 28. - Erosion on outside bend of conveyance channel below San Marcial, Middle Rio Grande Project.

Since this channel was much narrower than the normal river channel, it was anticipated the river would widen out to a width similar to the river. In this case, bank protection required varied from small jacks cabled together along the toe of the bank to a gravel blanket channel, figure 29.

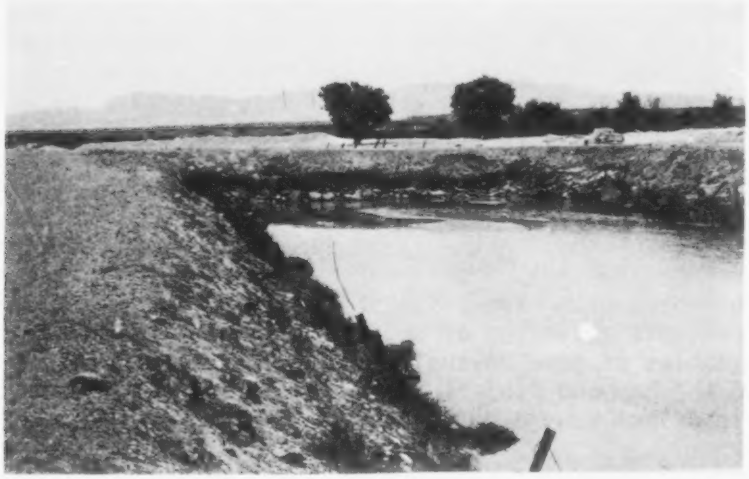


Figure 29. - Conveyance channel below San Marcial with jacks for bank protection, Middle Rio Grande Project.

Many canals, in crossing major rivers, are siphoned under these channels. The present design principle being used at these siphon crossings is to bury the siphon well below the maximum scour that would take place during a design flood peak. Figure 30 shows a sketch of a siphon and an

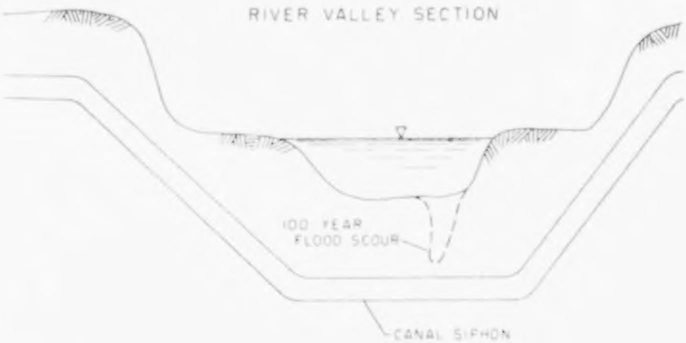


Figure 30. - Canal siphon under river channel.

estimate of the scour that would take place during a 100-year flood peak. Empirical relationships of scour are used for predicting this scour depth. As shown on the profile in figure 30, if the siphon were not buried a sufficient depth below the maximum scour, it would act like a small dam across the river. This would create a deep scour hole immediately below the siphon that could eventually undermine the siphon. At one time, siphons were not buried to the depths required or were backfilled with riprap or had a downstream sill that resulted in excessive downstream erosion. A photograph, figure 31, on Deer Creek in Central Valley, California, shows what can happen below a sill protruding above the siphon on the downstream side by about 3 feet. Another example, figure 32, where large floods undermine the siphon is shown for the Hammond Project where a siphon was actually undermined during a high floodflow.

Canals must also cross many smaller channels, and flow from these channels will be routed by use of culverts under the canal. The design of a culvert must consider handling the flood peaks as well as the sediment load. Some extreme problems of this nature have occurred where some unusually high floods with high sediment loads plugged the culvert. In these cases, reliable estimates of sediment movement and flood peaks are important. Examples of some unusual cross drainage problems are shown in photographs of the Hammond Project in New Mexico where culverts were plugged and floodflows caused damage to the canal (figs. 33 and 34).

A study program was initiated by the Bureau on the North Platte Project in 1960 to flush sediments from the Guernsey Reservoir to control excessive seepage losses from the project canals. The silts and clay sediments flushed from Guernsey Reservoir were carried reasonably well in the irrigation water throughout the entire length of the project canals and into parts of the lateral system and helped control excessive seepage.

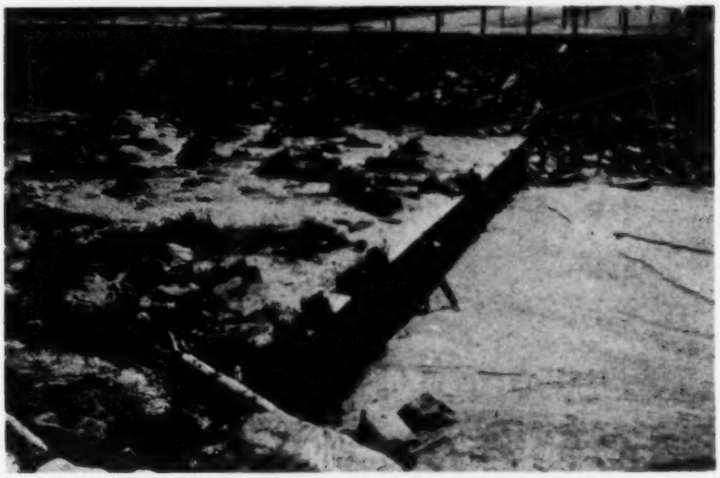


Figure 31. - Deer Creek siphon crossing on Friant-Kern Canal in Central Valley Project, California.

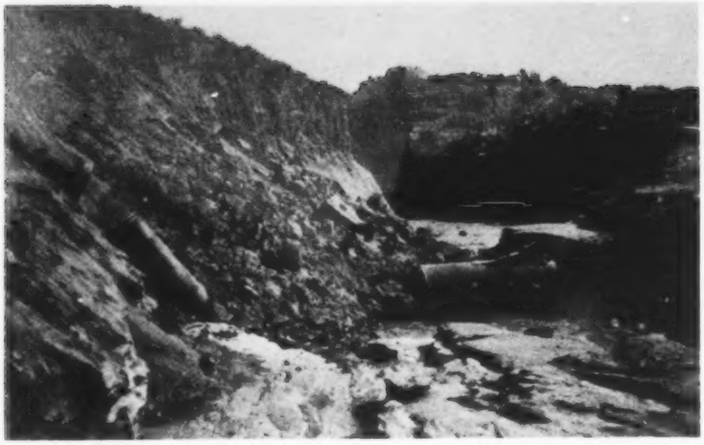


Figure 32. - Erosion of siphon crossing at Mine Canyon on Hammond Canal, Hammond Project, New Mexico, after September 1969 flood.

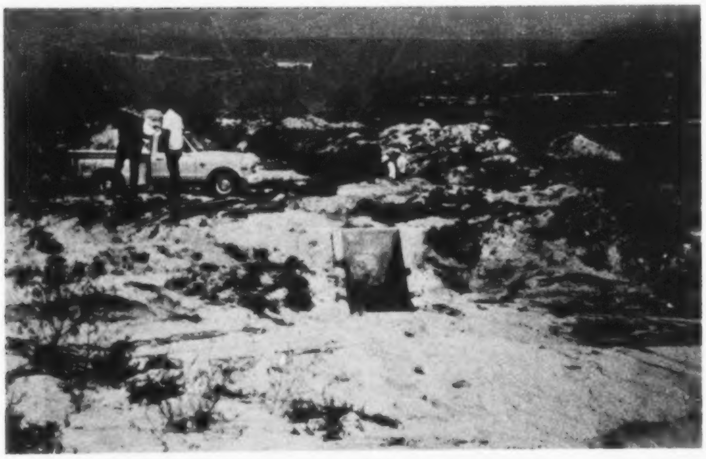


Figure 33. - Cross drainage channel culvert under canal on Hammond Project after September 1969 flood.

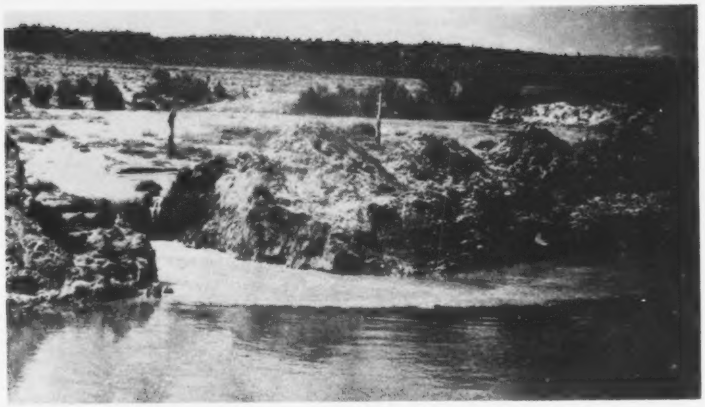


Figure 34. - Cross drainage flow overtopping canal bank on Hammond Project in June 1963.

Pumping Plants

The criteria for location of a pumping plant on a river are quite similar to those of a diversion dam. The best point to divert from a river is on a concave or outside bend and slightly on the downstream end of the bend. In the case of a pumping plant, it is even more critical to locate the diversion point on a bend because we have no control such as at a diversion with a sluice gate. An intake channel from the river to a pumping plant, figure 35, should be avoided because, as indicated previously, any takeoff from a channel at 90° will result in more sediment being moved into that channel than necessary.

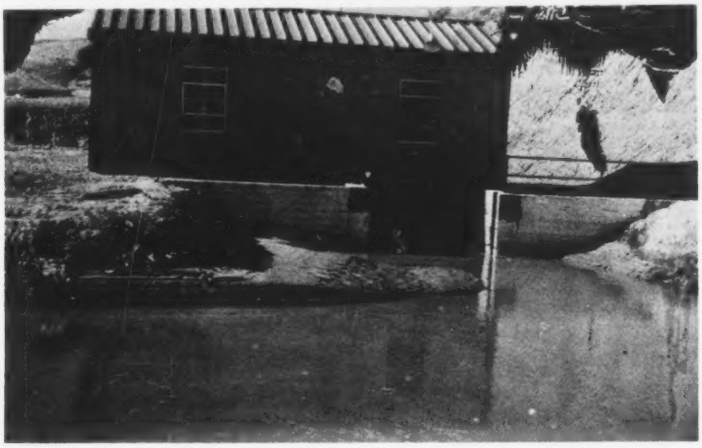


Figure 35. - Sediment deposits in intake at Savage Pumping Plant on Lower Yellowstone River.

Under the direction of Mr. W. M. Borland of the Bureau, a study was made of wear factors and their influence on pump capacities. This study showed general guidelines for increases in wear factors based upon the water-sediment complex. Table 1 shows the relationship as found by Mr. Borland.

Stability of Natural Channels to Convey Waste Flows and Return Flow

The channel stability balance (fig. 5) shows the unstable condition that can develop when water is added to a natural drainage channel from a canal wasteway or as return flows are added to a natural channel. As shown by the channel stability balance, an increase in water will create instability unless some modification is made either by change in sediment load or channel slope. A good example of this is the alkali drain on Shoshone Extension in Wyoming, figure 36, and on the Fivemile, figure 37, and Muddy Creeks in Wyoming above Boysen Reservoir. The solution to this instability can vary depending upon a cost comparison of remedial measures. Checks or control structures are examples of a technique for reducing the river slope. Another technique is that of riverbank protection to control excessive erosion because as flows increase, river channel velocities will increase and thereby cause scour.

Miscellaneous River Channel or Sediment Problems

In many Bureau project areas, the natural erosion taking place in river channels may endanger either irrigated lands or parts of the water distribution system. The Lower Yellowstone River near Sidney, Montana, is a good example of this problem. In this reach of the Lower Yellowstone River, ice conditions and natural bank erosion required some control to save the lateral system. Frequently, this excessive bank erosion may result from upstream changes and unstable downstream conditions, or it may be the natural movement of a river. There are many techniques for protecting banks against erosion. They are with steel jacks, rock groins, riprap banks, or other similar types of protection. The steel jacks were originally used in channels having high sediment loads in suspension because they reduce the velocities and cause deposition of sediment within the jack field. However, on many streams they are also used along the toe of a scouring bank in order to reduce the velocity of flow along this bank. Examples of the use of jacks are shown on the Rio Grande, on the Hammond Project in northern New Mexico, and on the Lower Yellowstone River (figs. 38, 39, and 40).

Groins protruding into the river with riprap or rock will also control lateral erosion of the banks. The photograph on the Lower Yellowstone shows a combination of jacks and groins. Riprap banks provide excellent protection. An example of this is on the Colorado River (fig. 41). Other combinations of rock, such as gabions or even the use of car bodies, will help control bank erosion. The basic criterion is that of providing a smooth, well-defined curve. In the case of car bodies that are not tied together, they may cause even worse damage by creating scour holes rather than smooth flow conditions.

Table 1. - Conditions of the water-sediment complex to be handled by pumping plants

WHERE FOUND	APPLICABLE TO	RECOMMENDED INCREASE IN CAPACITY (In %)	
		Small Pumps < 100 cfs	Large Pumps > 100 cfs
CLASSIFICATION: CLEAR WATER			
Reservoirs, large canals, drains, and some large snowmelt-fed streams	Reservoir, canal relift, drainage-type pumping plants, and branch from main canal	0-5	0-3
CLASSIFICATION: LIGHT SEDIMENT LOAD			
Small reservoirs, drains, canals, and snowmelt-fed streams	Reservoir, canal relift, drainage well-sump and river-type plants, and branch from main canal	5-10	2-5
CLASSIFICATION: MEDIUM SEDIMENT LOAD			
In few canals and drains that carry sediment-laden storm water frequently, and most rivers and streams where erosion is normal	A few canal relift and drainage plants, most well-sump types, at canal terminal, and some river-type plants	10-15	5-8
CLASSIFICATION: HEAVY SEDIMENT LOAD			
In rivers where erosion is great and aggrading sand streams	Well-sump and river-type plants and main canal terminal	15-20	8-15

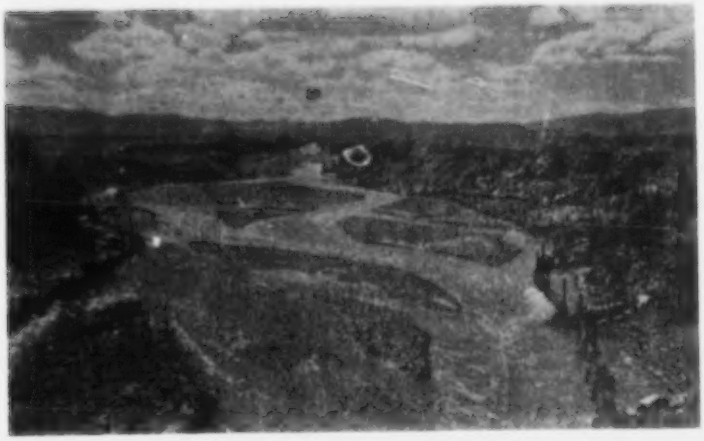


Figure 36. - Alkali drain on Shoshone Extension, Wyoming.

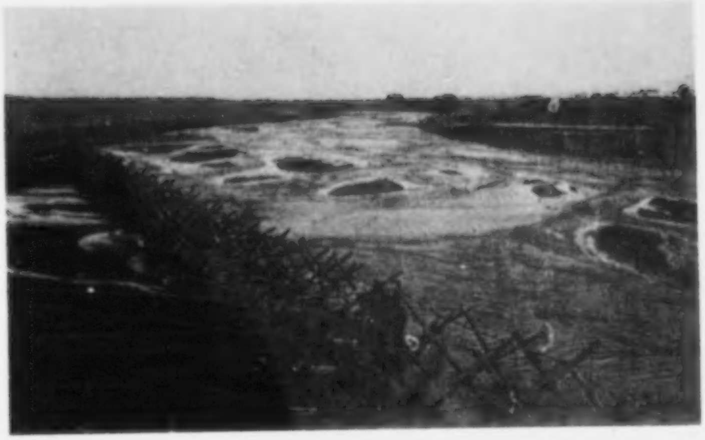


Figure 37. - Fivemile Creek on Riverton Unit, Wyoming.

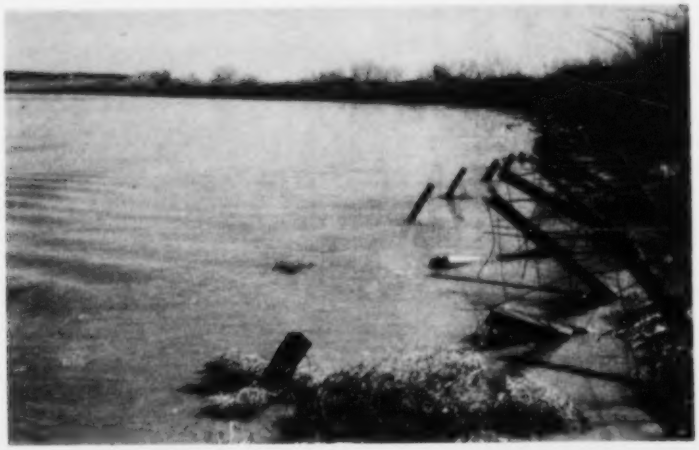


Figure 38. - Jacks used to control bank erosion on Rio Grande near Albuquerque, New Mexico, in April 1962.

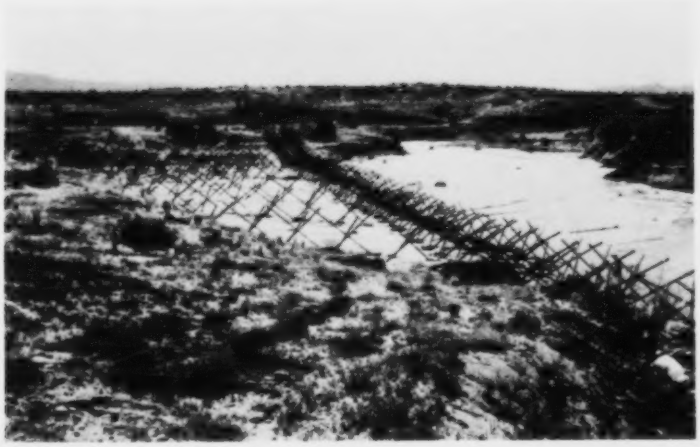


Figure 39. - Jacks used to control channel meander on cross drainage channel on Hammond Project, New Mexico.

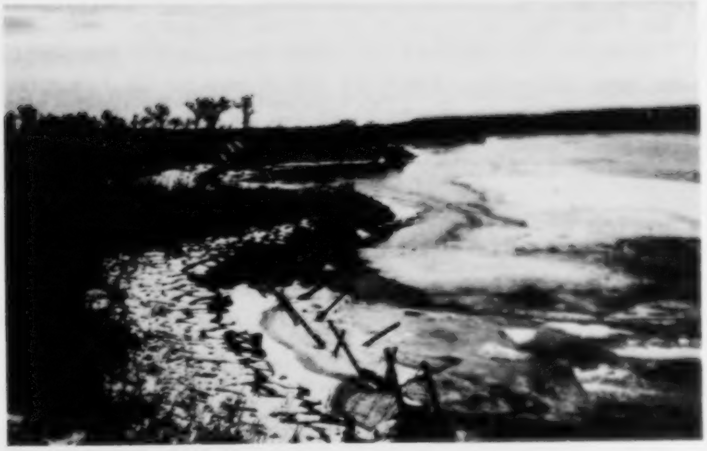


Figure 40. - Downstream view of rock groins and jacks on Yellowstone River, Lower Yellowstone Project, Montana.

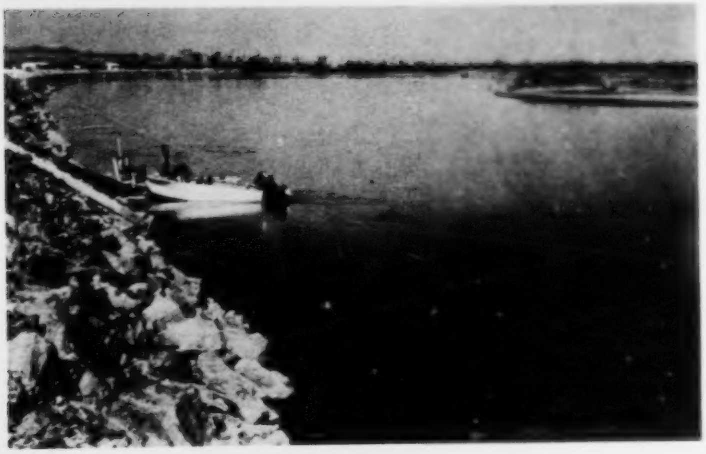


Figure 41. - Colorado River bank line protection with riprap.

The change in flow pattern of a stream can also result in an unbalanced condition. Releases of water below a storage dam into a natural channel for eventual diversion downstream can upset a fairly stable channel regimen. Although the dam will control flood peaks, the releases for downstream diversion represent an increase in percent of time the river channel will be flowing at a constant flow somewhat less than bank-full stage. This increase in percent of time the banks are wetted may cause an increase in bank erosion, such as experienced on Frenchman Creek in Nebraska in the approximate 51-mile reach below Enders Dam and above Culbertson Diversion Dam. Steel jacks are being used to reduce the velocities along the banks, and thereby reduce the bank erosion.

The abrasive action of sediment consisting of the more angular sand sizes can be extremely damaging. Sand material in suspension eroded away the concrete in the right diversion tunnel at Glen Canyon Dam during the construction period, leaving the aggregate exposed as shown in figure 42.

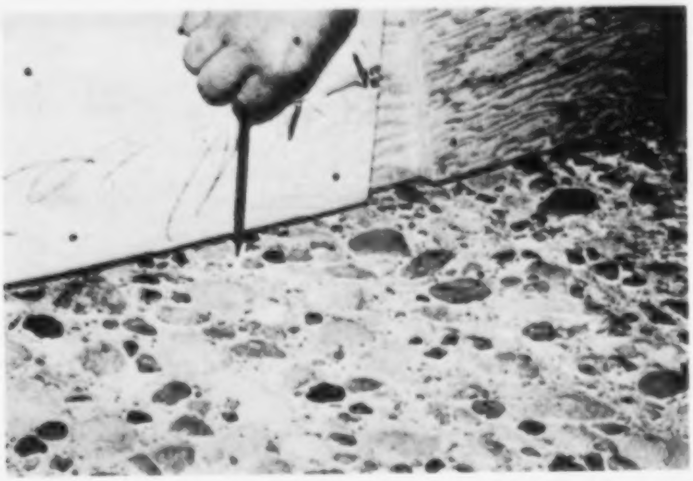
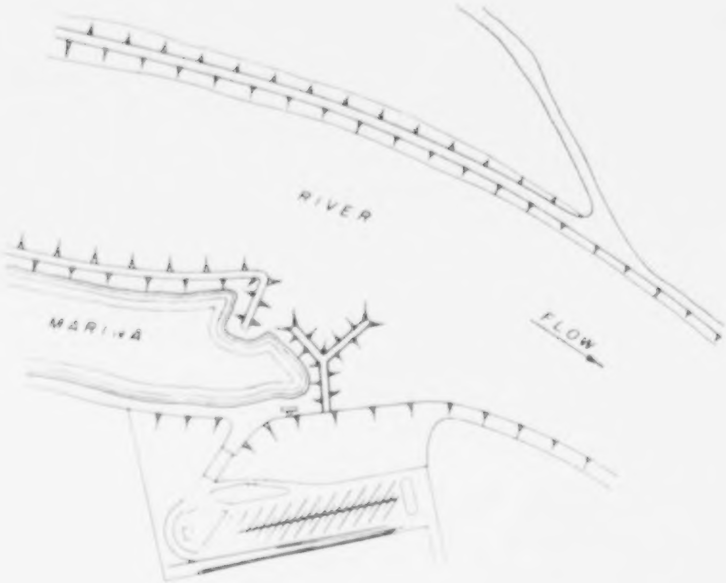


Figure 42. - Erosion of concrete in right diversion tunnel at Glen Canyon Dam.

Although not directly related to a pumping plant, a similar problem is the design of an intake from a river such as an entrance into a marina or backwater area. Although the Bureau has not model-tested this problem, tests were made by the Corps of Engineers in their laboratory near Omaha, Nebraska, of an intake channel for reducing sediment deposition at the entrance. The design principle, in this case, was to break up the sandbar that would form in the intake channel and thereby maintain an adequate entrance. Figure 43 is a sketch developed from the model studies made by the Corps of Engineers. The Bureau has constructed an entrance, somewhat similar to that shown in figure 43, on the Colorado River at the marina entrance near Needles, California, figure 44. This has worked satisfactorily.


RIVER
MARINA
FLOW

.Figure 43. - Example of boat marina entrance.

Figure 44. - Marina on Colorado River at Needles, California.

Conclusion

Many of the sediment problems requiring a continuing operation and maintenance program cannot be avoided. However, the Bureau has found that, with a better knowledge of the sediment transport concepts and river channel characteristics, many of these operation costs can be reduced. Changes in design techniques such as in adjustments to diversion by use of sediment excluders, design of siphon crossing of natural rivers for passing floods, and bank protection for natural channel scour due to the instability of river channels are examples of methods being used to reduce some of the operation and maintenance costs.

Water Systems Operation and Maintenance Workshop - 1990

MOBILE EQUIPMENT MANAGEMENT, SELECTION, AND USE

Session 5-1

Merle Gibbens

Some of the most important decisions which can be made by an irrigation district or canal company are those which have to do with equipment selection, procurement, and use. These are decisions that the irrigation manager, board members, and staff must make on a recurring basis or at least as often as annually at budget time.

Equipment requirements are determined by the size of the project, type of water delivery, facilities, terrain, climate, and to some extent, available personnel. Only people thoroughly familiar with these facts can intelligently make the proper equipment requirement determinations. Work requirements on an irrigation district change, new equipment becomes available, and the proper mating of these two things is what determines the value of the decisions involving equipment needed.

One of the most difficult decisions to make is that of determining when the remaining life of a piece of equipment can no longer justify additional repairs. In reaching this decision, some of the factors to be considered are undepreciated value, cost of repairs, estimated remaining hours of use, and future maintenance and rehabilitation programs. There are other considerations to weigh before replacing equipment. Whether or not similar or adequate equipment is available under equipment rental is one of the considerations. Whether or not the work can be economically performed by contract is another consideration. In some areas, equipment rental is not practical either because the particular piece of equipment is not available or because it may not be available in cases of emergency Many pieces of equipment which managers once thought had to be owned by the project can now be obtained by equipment rental. Some of these pieces of equipment are concrete mixers, road graders, scrapers, and compressors. There is always the question of when ownership is more economical than equipment rental. Many operators feel that 300 hours of annual use is the approximate point where ownership is more economical.

There are certain points to consider in equipment rental. Some of these are:

1. With or without operator
2. Who furnishes fuel and lubrication
3. Cost per hour of operating
4. Operating time per week or day
5. Cost per hour of standby time
6. Transporting equipment to and from jobsite

Another means of getting certain types of maintenance work done that is closely related to equipment rental is by contract. The major difference between the two is the unit of measure. The contractor under a contract bids so much a yard for riprap placed in a lateral or yard of gravel placed on an operating road. With all contracts, it is very important to have positive control of the unit of measure, whether it is lineal feet, cubic yards, or miles.

It is good management to have a complete inventory of contract services and equipment available for rent in your area. Utilizing these services can result in reducing your fleet and staff while properly maintaining your project at less cost. On occasion there has been an exchange of personnel and equipment between entities such as Bureau of Reclamation, Forest Service, Bureau of Land Management, irrigation districts, and highway districts - a cooperative spirit of "you scratch my back and I will scratch yours."

Under certain programs such as a small rehabilitation program or a program where the hours of annual use are close to the breakpoint of equipment rental, it may be more economical to purchase used equipment. Here you need the assistance of your mechanic and operator as well as the services of a good, reliable dealer. It will be worthwhile to consult with the original owner of the used equipment if this is at all possible.

Let us now consider some of the factors in the selection of the various categories of equipment. When we consider the selection of vehicles, we cover a wide range of equipment ranging through cars, trucks, and heavy equipment.

Many utility companies are replacing a large part of their standard passenger cars and light pickups with the compact-size vehicles. These vehicles are used mostly in supervisory or light maintenance work. Recent survey by these companies revealed that the changes most desired in compact vehicles include:

1. Larger wheels and tires
2. Increased durability
3. Higher horsepower
4. More durability in brakes, clutches, engines, front ends, and transmissions

All of these additions mean that the compacts stand to lose or at least lessen their cost advantage. The consensus of these companies was that as long as the vehicles are not subjected to rugged use, the lower initial cost and lower operating cost of a compact make it an economical vehicle to own. Applying the experience of these utility companies to water projects indicates that the compact is suitable for operation and supervisory purposes. Where these vehicles are used for both operation and maintenance, the 1/2-ton and 3/4-ton sizes are probably more economical because of reduced maintenance costs. The extras available on these pickups (fleetwing body, heavier springs, four-speed transmission, and positraction rear end) are worth considering because of the reduction of maintenance costs and increased flexibility. Do not overlook automatic transmissions, particularly in ditchrider vehicles where there are many stops and starts. A few years ago there was a great deal of driver resistance to the 3/4-ton pickup. The manufacturers have eliminated much of the hard riding and difficult steering that were the main objections to this size of vehicle.

A few years ago, the basic truck on water projects was the 1-1/2-ton flatbed or dump. Many projects are replacing these with the 2-ton dump or flatbed. They are purchasing a truck with heavier front springs, two-speed axles, and power steering. These larger units have the ruggedness to economically perform maintenance work with less maintenance cost than the 1-1/2-ton unit. Some projects have used the side-dump flatbed rather than the rear-dump flatbed. They have found that extra care has to be used in balancing the load, particularly if there is only one hydraulic ram. Those who have used the side dump recommend two hoists. This will eliminate the twist in the flatbed resulting from an unbalanced load. Four-yard trucks that side dump are about a maximum size for irrigation district use.

The need for the larger trucks will vary among projects. Unfortunately, many projects are severely limited in the size of equipment they can operate because of poor O&M roads. In the selection of larger units, it is important to become familiar with the many extras available and with the relationship of the additional costs of these extras to the increased performance and the reduced maintenance. There is nothing more important in the selection of your truck than the proper balance of all the components in your vehicles. The larger the unit, the more important this becomes.

Let's now consider some of these components. Of first consideration is the engine - should it be gasoline or diesel? The diesel engine will save 10 to 20 percent on fuel and 30 percent on maintenance. Therefore, the more miles and more capacity loads, the less operating cost per mile. Diesels will also idle on about one-fourth the gallons used by gasoline engines. But this type of use adds up to miles. There is no question that the diesel is most practical in operations above 40,000 to 50,000 miles per year. With less miles per year, the gasoline-powered truck becomes more economical. A gasoline-powered truck will cost approximately $2,000 to $3,500 less than a comparable diesel model. In addition, there are savings in interest, insurance, and license fees. Other advantages in the gasoline engine are:

1. Lighter weight
2. Lower overhaul costs
3. More optimistic resale price

The most common transmission and axle combination in the 1-1/2-ton truck is the five-speed transmission with the single-speed axle. In the 2-ton size, the five-speed transmission is in combination with the two-speed axle. As truck sizes increase to the larger 3- to 5-ton trucks, the five-speed transmission is joined with three-, four-, or five-speed auxiliary transmission and a single-speed axle to give the necessary steps and power to handle the loads.

At some point when you purchase the larger units, because of highway restrictions, you will be forced to consider tandem axles. There are three different designs of the single-speed tandem axle:

1. Both axles geared to drive.

2. One axle geared to drive with sheaves mounted next to wheels which, through belts, drive a following set of wheels. (It appears that the trucking industry prefers the "geared" axle to be in the rearward position in order to obtain a longer drive shaft, thus reducing the critical nature of the angle caused by short ones.)

3. One axle geared to drive, with the second axle employed as a weight-carrying member only.

In addition, there are important advantages inherent in the tandem axle design:

1. Greater safety. There are six wheels on which brakes are mounted instead of four wheels on a two-axle tractor. This feature provides 28 percent more brake lining area on the tractor.

2. The suspension is usually of the "walking beam" type which either utilizes a nonflexible member or a spring as the equalizing beam; reduces road shock by 50 percent.

In these heavy-duty units, it may be best to go to the larger front axle. The small axles are not as safe and the extra wear on bearings, bushings, kingpins, and tierods under overload conditions could prove more expensive than that of the heavier axle.

As in pickups, do not overlook automatic transmissions in the larger units. This is a very important consideration if you do not have skilled truck drivers or where your trucks are occasionally being driven by temporary, unskilled employees.

The brake safety and performance requirements as established by Federal, state, and municipal authorities should also be reviewed in detail. Individual states have important specific brake regulations. The principal components to be considered in a brake selection are:

1. Brake system
2. Foundation brakes
3. Brake drums
4. Parking brake
5. Safety brakes
6. Retarders

The vacuum booster-hydraulic brake system is a type of operation combining manual or physical power and engine manifold power, an "assistor" type system. A full-power system for brake operation is conventionally an air system. This full-power system has found almost complete acceptance on diesel-powered straight trucks and combination vehicles.

Development of brakes with automatic lining-drum clearance mechanism is an accomplished design and is being marketed for brakes on both light- and heavy-duty vehicles. In addition to eliminating the service operations, the automatic adjustment maintains a constancy of clearance which assures uniform brake timing and power, thereby providing the economies in brake operation obtained through good brake balance. Safety devices such as "springset" units can be added.

Item 3 touches on a component offering the advantage of light weight with the accompanying disadvantage of either high initial cost for an aluminum drum or higher brake operating temperature which invariably results in lower lining mileage.

For item 4, the choice on light trucks is between a transmission-mounted propeller shaft brake and a mechanical pickup with the brakes at the rear wheels. Both systems are widely used. The transmission mounting enjoys the power advantage of the rear-axle ratio and simplicity of hookup. It also provides a completely independent secondary brake system which is not affected in its performance by the variable lining-drum clearance of the service brakes.

Regulations in most states forbid the use of other than mechanical power to apply the rear brakes for parking purposes because of "leakoff" possibilities. Most use rear brakes for both "parking" purposes and "emergency" stopping requirements. This trend has resulted in the adoption of so-called safety brakes or "springset" brakes, which are item 5. These devices are applicable to full-powered, air-operated brakes and function to remain in a released or inactive position, while air pressure to the spring compressing mechanism results in a brake application bv spring force.

Automatic brake adjusters to insure uniform lining-drum clearance are, therefore, a prime requisite for optimum performance of rear-wheel parking brakes, either spring or mechanically operated.

Item 6 indicates a consideration of "retarders" for vehicle braking. The increasing use of automatic transmissions in light trucks makes a retarder consideration a reality by the introduction of this device in the torque converter section. The weight and cost penalties are minimized in such a construction, and the unit will be found adequate and effective for brake requirements in traffic as well as downhill.

The trend in heavy-duty trucks toward the use of brake retarders has been in the area of exhaust and engine brakes. Although these enjoy a weight advantage over other types, they are also somewhat limited in the braking capacity when compared with fluid or electric retarders.

Certainly, they should be of the self-adjusting type to reduce maintenance labor. The very fact that the lining-to-drum clearance remains constant throughout the lining life will result in minimum air usage and better brake timing.

When one starts thinking about the electrical system, alternating-current-type generators are now standard in all vehicle specifications because of their capability to deliver a charge rate at engine idle speed and also to perform well in the higher speed ranges. To complement the long-life design alternators, full transistor regulators are recommended. These do not contain any moving parts that deteriorate, such as contact points; have no conventional source for radio interference, and thereby improve the range and quality of two-way communications; and are not affected by ambient temperature. Recommend 65-70 amp capacity alternators.

Much of the maintenance work on the canals, laterals, and drains on a water project is done by draglines, Drott Yumbos, backhoes, Gradalls, etc. These are very important pieces of equipment on a water project. The size of this equipment is based upon the size of the canal or drains to be cleaned and the necessary reach to do this job. Often there is a tendency to purchase a larger piece of equipment than is necessary because of the greater earth-handling capacity. If you are involved in long range construction or rehabilitation, the greater earth-handling capacity should be considered, but this does not necessarily hold true on normal maintenance. Even with smaller machines, you will notice that the buckets are seldom filled. In addition, particularly in the maintenance of drains, the larger machines and buckets overexcavate which results in reduced water velocities which tend to build up deposits of silt and encourage growth of weeds.

Dragline buckets are manufactured in the light-, medium-, and heavy-duty weights. In normal maintenance, the light- or medium-weight bucket will provide many years of satisfactory service. There are also perforated buckets that are highly desirable for maintaining canals and drains. The condition of your maintenance roads and bridges, as well as state highway weight restrictions, may limit the size of the machines. The size of your hauling equipment or the cost of renting heavy hauling equipment imposes a limitation on the size of the machine. Hauling equipment is usually available that will handle the 1- to 1-1/2-yard machine. Machines above this size are difficult to justify because of the high moving costs.

Closely related to the dragline is the hydraulic-controlled excavator such as the Gradall. These machines have positive control booms and buckets. There are numerous types and sizes than can be purchased for specific jobs. Good types of similar machines are like the Drott Yumbo, Hop To, etc., which due to the type boom (not telescoping) can excavate with more latitude in the placing of the carrier. These machines are not a high-production piece of equipment in terms of cubic yards moved per hour. Their value lies where positive control of the bucket is needed such as in cleaning a lined lateral or canal, cleaning around structures, or for fine grading in placing structures. When this machine is to be used to clean canals or laterals, it is preferable to purchase the equipment with a torque converter in the truck drive. This is of great assistance to the truck drivers; otherwise, they would have to press down and release the clutch three or four times a minute. For a cleaning job that is more or less continuous, both a truck driver and an operator are required. This will require some type of communication between the truck driver and operator to coordinate the moving with the operating. If a lot of your work is spotty or in short stretches, consideration should be given to a remote control unit where the operator controls the truck from the machine cab. This unit is also available for draglines.

There are a group of machines that, in the last few years, have been used on nearly every project. These are the loader-backhoe combinations. They are highly mobile units with many uses. The loader and backhoe unit is hydraulically controlled and mounted on an industrial farm tractor. The larger units approach the Gradall in performance and are considerably less expensive, and also more versatile. If you are planning on using the loader unit to excavate and load material lying in its natural state, the loader capacity should be at least 1-1/2 cubic yards. Another attachment for this machine is the "4-in-1" front-mounted bucket. This bucket serves as a shovel-dozer-scraper or clamshell.

Another basic unit on a water project is the crawler-tractor. On most projects it is used for bulldozer work, leveling spoil, building roads, and repairing canal breaks. It is also used for operating the side sloper or pulling heavy-duty brush cutters. If the tractor is to be used for operating the side sloper, it is important to have the proper size tractor for the sloper. Many improvements have been made by the manufacturers to reduce daily servicing time costs in these units. Some of these improvements are electric starting, sealed track rollers and idlers, and hydraulic-type track adjusters. However, even this unit is being challenged by smaller rubber-mounted units stressing mobility.

Properly maintained O&M roads are very important to the operation and maintenance of the project. This maintenance can best be done by a motor grader. It is very difficult to justify the ownership of this equipment on small projects. Justification is made easier if some of the lateral and canal cleaning can be done with the sloper attachment to the grader. Equipment rental or contract is probably the most economical way of maintaining these roads on the smaller project.

Do not overlook office equipment when reviewing equipment requirements and replacements. Many administrative costs can be sharply reduced by replacing obsolete and worn out office equipment. There have been many improvements in billing and cost account machines, duplicating and reproduction equipment, self-answering and recording telephones, and other office equipment that will increase the efficiency of the administrative section. Electronic calculators are very reasonably priced, fast, and dependable.

Mobile radio equipment is being used on an ever-increasing number of projects. Improvements in these radios have greatly reduced the unit maintenance costs. Increased efficiency in operation, water utilization, maintenance, and the correlation of all project programs with the use of this piece of equipment have certainly removed it from the class of an expensive toy. Closely related to the communication equipment are the new developments of supervisory radio controls. Some projects are now dispatching water and providing additional protection to their projects with this equipment. Supervisory radio control can provide additional safety to your system, increased efficiency in water utilization, and decreased cost of operation. Project managers should consider this type of equipment to determine whether or not it can be efficiently used on their projects.

Some projects, no doubt, will want to consider the practicability of snow machines and motorcycles. These machines have been perfected and can be depended on to provide cheap, quick transportation, where roads may not be usable or nonexistent, in the gathering of watershed data and operation duties in remote areas. Planes or helicopters, usually on a charter basis, may also fill a requirement in this regard.

There are many other pieces of equipment necessary for the proper operation of a water project. Whether the piece of equipment costs $1,000 or $140,000, the same factors should be considered. These factors are repair versus a new replacement; equipment rental versus ownership.

Most projects purchase equipment by preparing and issuing specifications covering the general size and type of equipment that they wish to buy. This can result in a lower initial cost of the equipment. Specifications should list basic requirements of components and the general performance required. The type of specifications that will permit at least four to six bidders will generally make it possible to purchase a satisfactory piece of equipment at the most economical cost. This does not mean that the least expensive piece of equipment has to be or should be selected. However, this will provide the project manager with a basis for comparing comparable equipment offered by various manufacturers. Project managers should not overlook adequate warranties on new equipment and they should insist on as much protection as possible.

When the time comes to upgrade equipment, some projects are finding there is money to be made in auctioning off the old equipment to the highest bidder rather than trading it in to the dealer. In some instances, used equipment that is well maintained is bringing in four times as much at auction as the dealer's best offer. In addition to this benefit, many times a better buy can be made from the dealer if there is no trade-in. Depending on the local situation, of course, the possibility of auctioning off the used equipment probably should be given careful consideration.

In the past, service and preventive maintenance programs have many times been delegated to subordinate levels within many organizations. Technology and manufacturing innovations have provided us the equipment and tools to solve the problems of material movement, distance, and time. As a result of these evolutions, equipment costs, equipment scheduling, and related maintenance represent an increasingly larger proportion of the total operating budget. Equipment management and preventive maintenance is of vital interest to all levels of management.

The "market value" of most water user organization's equipment fleet make "seat of the pants" management technique unreliable. Many maintenance functions must be performed and management decisions must be made in the life time of most items of equipment that some form of record system becomes necessary. The administration of the record system will be dictated by local needs and conditions. Types of systems range in sophistication from notebook systems to card file systems to complex computerized systems. When setting up or administering an equipment record system, the key point to remember is that the system is a tool toward efficient management. The record system must not be allowed to become a burden in itself.

A successful record system will become an invaluable tool in scheduling, preventive maintenance functions, predicting and preventing breakdowns, and expediting repairs. Such a system will help plan future equipment budgets and estimate job costs. A well designed and administered record system will answer such specific questions as: How much does downtime cost? Which machine is most economical for which job? How can breakdowns be minimized? What are the most economical repair methods? Who should be responsible for what inspection and maintenance functions? Is renting better than owning? Should it be repaired or replaced? What types of fuels and lubricants are best? What types of tires are best? At this point, it would be easy to conclude that a record system that gives so much information would be burdensome. It need not be.

Start with preventive maintenance. A checkoff system to keep track of service intervals, so that filters are changed periodically and lubrication is performed on schedule, is one method used. Incorporate a flag system to the machine record so that components with long service intervals are not forgotten.

Complete cost data should be recorded from the original purchase to the final disposal. The data should include three major areas of expense: items consumed in routine preventive maintenance, repair parts, and labor. It is important to distinguish between routine service and unexpected repairs. Preventive maintenance such as filters and periodic maintenance such as linings can be scheduled with reliable accuracy; thereby, facilitating cost comparisons between similar vehicles or machines. Sudden failures of working parts are not so predictable and also are not necessarily valid comparisons to other similar machines. You cannot escape the fact that normal maintenance costs rise as a machine is used. The goal is to derive the maximum benefit from the machine while it is economical to do so and to dispose of it at the time it becomes cost prohibitive in comparison to available alternatives.

One example of a recordkeeping system is basically a card file system. It accomplishes three basic functions:

1. Management of preventive maintenance and repairs by shop and operating personnel

2. Documentation of job costs

3. Administration of an effective equipment replacement program

Documenting repair costs is only one component of total equipment cost. It is also necessary to know actual operating costs to evaluate machine performance per a specified measurable unit in order to determine job equipment costs.

Maintaining a fuel issues sheet at each fuel service island will facilitate documentation of fuel and oil consumption between service jobs. The data will also be useful in auditing monthly fuel inventory balances.

In addition to shop personnel, operators should be involved in the maintenance program. A good operator performs preventive maintenance all day long. Normally, the operator should perform daily service functions and record the same, and also should keep shop personnel informed of the general condition of the machine. A service-conscious operator will identify early warning signs and will take proper remedial or corrective action to safeguard against major equipment breakdown.

Readily available resource materials and training are the key to building a conscientious staff. See that appropriate manuals for operation, parts, repairs, and service are supplied by equipment vendors. Most employees respond enthusiastically when they see that you care enough to keep them informed of new ideas and advancements in the mechanical repair industry. Some manufacturers will send factory personnel to your organization for staff training purposes. The services are invaluable when they can be arranged.

The following forms for documenting equipment repair and operating costs are "example" forms which may be useful in establishing or improving your present recordkeeping system:

FORMS

Operator's Daily Shift Ticket
- Provides daily check of machine's condition
- Record of daily operating time
- Provides early detection of major repair needs or scheduling

Motor Vehicle Service and Inspection Report
- Physical check of vehicle components at service time
- Good safety check of vehicle

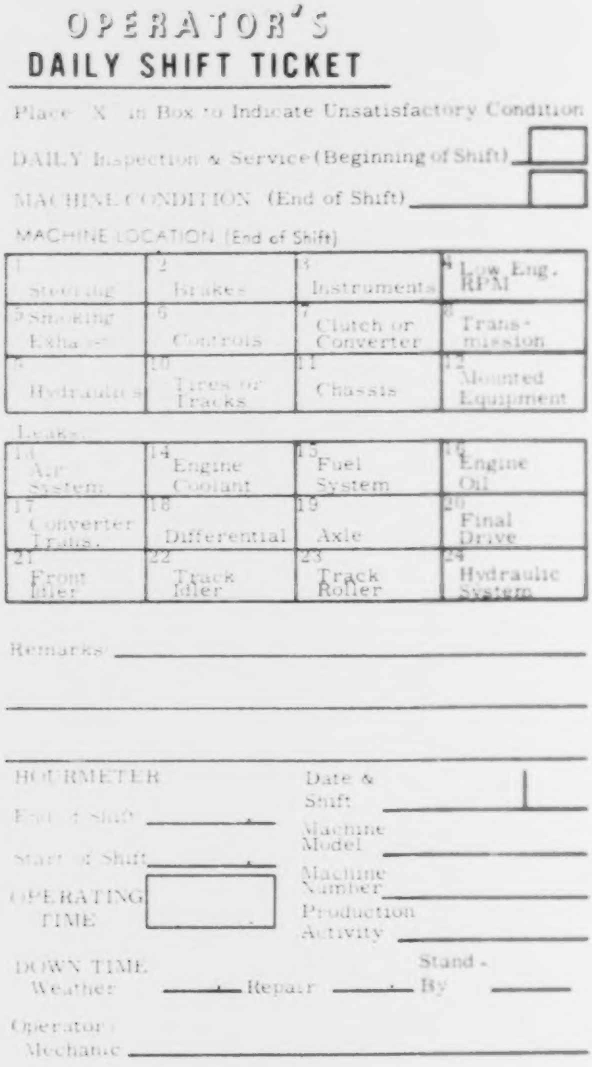
Motor Vehicle, Machinery and Mechanized Equipment
- Inspection Report
- Scheduled periodic inspection

Mechanic's Daily Labor Report
- Record of mechanic's time for performing specific vehicle or equipment repair

Shop Order No.
- Documentation of total repair costs
- Excellent summary for capitalizing major overhauls

Gasoline Issues
- Issue can be expanded to segregate gasoline or fuel types; i.e., unleaded, diesel, propane
- Sheet should be available at fuel island

Equipment Operation and Maintenance Costs
- Other measurable units are hours
- The particular equipment cost per hour or cost per mile will facilitate job cost estimating, equipment repair/replacement scheduling, and equipment scheduling.
- Often, form is expanded to include annual depreciation expense



OPERATOR'S
DAILY SHIFT TICKET

Place "X" in Box to Indicate Unsatisfactory Condition

DAILY Inspection & Service (Beginning of Shift) ☐

MACHINE CONDITION (End of Shift) ☐

MACHINE LOCATION (End of Shift)

1 Steering	2 Brakes	3 Instruments	4 Low Eng. RPM
5 Smoking Exhaust	6 Controls	7 Clutch or Converter	8 Trans-mission
9 Hydraulics	10 Tires or Tracks	11 Chassis	12 Mounted Equipment

Leaks:

13 Air System	14 Engine Coolant	15 Fuel System	16 Engine Oil
17 Converter Trans.	18 Differential	19 Axle	20 Final Drive
21 Front Idler	22 Track Idler	23 Track Roller	24 Hydraulic System

Remarks: ______________________________

HOURMETER

End of Shift ________

Start of Shift ________

OPERATING TIME ☐

Date & Shift ________

Machine Model ________

Machine Number ________

Production Activity ________

DOWN TIME Weather ________ Repair ________ Stand-By ________

Operator / Mechanic ______________________________

MOTOR VEHICLE SERVICE & INSPECTION REPORT

Date:

Identification No.:

Kind of Equipment:

Make:

Model:

Capacity:

Speedometer Reading:

First Aid Kit: ☐ Full ☐ Empty ☐ None

Fire Extinguishers: ☐ Full ☐ Empty ☐ None

Miles Operated During This Period:

CHECK DEFICIENCIES NOTED BY CODES: "O" OPERATING - "I" INSPECTION ON APPLICABLE ITEMS BELOW.

Item No.	ITEM	O.K.	Adjust	Repair	Item No.	ITEM	O.K.	Adjust	Repair	Item No.	ITEM	O.K.	Adjust	Repair
1	Service Brakes				14	Generator - Regulator				27	Differential			
2	Emergency Brakes				15	Ignition - Starter				28	Tires (Indicate by No.)			
3	Head & Tail Lights				16	Distributor - Carburetor				29	Steering Gear, etc.			
4	Clearance Lights				17	Fan & Water Pump				30	Muffler - Exhaust Pipes			
5	Reflectors				18	Fuel Pump - Lines - Tank				31	Cab-Body-Frame-Fenders			
6	Wiring (General)				19	Instruments - Dash				32	Stake-Dump Bed-End Gate			
7	Horn, Bell, Whistle				20	Battery - Cables				33	Winch - Hoist - Cables			
8	Windshield & Wipers				21	Schock Absorbers				34	5th Wheel - Power Takeoff			
9	Glass (General)				22	Springs - U-Bolts				35	Wheels-Wheel Bearings			
10	W/S Washer Unit				23	Clutch-Linkage				36	Front-Rear Axle Assys.			
11	Rearview Mirrors				24	Transmission				37	Flares, Flags, etc.			
12	Radiator				25	Transfer Case				38	Brake Hoses-Connection			
13	Motor (General)				26	Drive-Shaft-U-Joints				39	Breather-Air Cleaners			

RECORD OF SERVICING

(INDICATE SERVICE PERFORMED VEHICLE BY SPEEDOMETER READINGS)

Oil Change	Lubricate	Battery	Washed	Brakes-Shocks	Transmission	Differential	Wheel Bearings

Have Previous Suggested Repairs, Replacements, or Adjustments Been Made: ☐ YES ☐ NO

Are Operating Rules Posted: ☐ YES ☐ NO

In Your Opinion is Equipment in Safe Condition: ☐ YES ☐ NO

REMARKS:

Project or Individual Responsible For Equipment:

(Signature of Inspector)

MOTOR VEHICLE, MACHINERY & MECHANIZED EQUIPT
INSPECTION REPORT

AREA:

PROJECT:

DATE

KIND OF EQUIPMENT:

MAKE:

MODEL:

CAPACITY

SPEEDOMETER READING:

APPROX. HOURS OPERATED

IDENTIFICATION NUMBER

CHECK CONDITION OF ITEMS APPLICABLE TO EQUIPMENT BEING INSPECTED

ITEM NO	ITEM	GOOD	FAIR	BAD	NONE	ITEM NO	ITEM	GOOD	FAIR	BAD	NONE
1.	Tires (show number in each condition)					26.	Cables				
2.	Foot Brakes					27.	Cables Sheaves				
3.	Emergency Brakes					28.	Boom				
4.	Steering Device					29.	Dipper Stick				
5.	Head Lights					30.	Track Rollers				
6.	Tail Lights					31.	Track Assemblies				
7.	Clearance Lights					32.	Drive Sprockets				
8.	Reflecters					33.	Drive Chains				
9.	Horn, Bell, Whistle or etc.					34.	Open Gear				
10.	Windshield					35.	Elevator Belting				
11.	Glass					36.	Cutting Edge				
12.	Windshield Wiper					37.	Mould Board				
13.	Rear View Mirror					38.	Instrument Panel				
14.	Flares					39.	Gauges				
15.	Flags					40.	Battery				
16.	Motor (General)										
17.	Generator										
18.	Wiring										
19.	Body										
20.	Cab										
21.	Stake Bed										
22	Dump Bed										
23.	Hydraulic Lift										
24.	Other Lifting Device										
25.	End Gate										

FIRST AID KIT ☐ FULL ☐ EMPTY ☐ NONE

FIRE EXTINGUISHERS ☐ FULL ☐ EMPTY ☐ NONE

HAVE PREVIOUSLY SUGGESTED REPAIRS OR REPLACEMENTS BEEN MADE ☐ YES ☐ NO

ARE OPERATING RULES POSTED ☐ YES ☐ NO

INSTRUCTIONS

This form will be used for inspections in compliance with requirements set forth in "Safety Requirements", Par. 8318.03 Motor Vehicles and Par. 8316.01 Machinery and Mechanized Equipment Inspection.

REMARKS

DATES OF WEEKLY INSPECTIONS

MILES TRAVELED DURING MONTH

GASOLINE USED

OIL USED

IN YOUR OPINION IS EQUIPMENT IN A SAFE OPERATING CONDITION ☐ YES ☐ NO

SIGNATURE AND TITLE OF PERSON MAKING INSPECTION

MECHANIC'S DAILY LABOR REPORT

Date ________________

Hours	Shop Order No.	Vehicle or Equipment No.	Description of Work

SHOP

DATE

DESCRIPTION OF WORK

SHOP ORDER NO.

MILES or HOURS

MAKE & MODEL

VEHL or EQUIP NO

ACCOUNT NO

MATERIALS

Description or Doc. No	AMOUNT
OPERATING	
Total Oper. Mat	
MAINTENANCE	
Total Maint. Mat.	

LABOR

Name	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	Total Hours
OPERATING LABOR																																
TOTAL OPERATING LABOR																																
MAINTENANCE LABOR																																
TOTAL MAINTENANCE LABOR																																

SHOP FOREMAN

GASOLINE ISSUES

____
(week or month)

Gas (gal)	District No.	Miles	Non-Vehicle Use	Signature
			Purpose	

DO NOT WRITE BELOW THIS LINE!

Subtotal

Certified correct

Total issues this period ____________

EQUIPMENT OPERATION AND MAINTENANCE COSTS

REPORTING PERIOD ____________________

Vehicle number	Type	Year	Date acqu-ired	Mileage to date	Operating cost	Mainte-nance	Miles used this period	Fuel	Miles per gallon	Cost per mile
1-41	Chev 1/2	78	3/78							
1-42	Chev 1/2	78	3/78							
1-43	Chev 1/2	78	3/78							
1-44	Chev 1/2	78	3/78							
1-45	Chev 1/2	78	3/78							
1-46	Chev 1/2	78	3/78							
1-47	Chev 1/2	78	3/78							
1-48	Chev 1/2	78	3/78							
1-49	Chev 1/2	78	3/78							
1-50	Chev 1/2	78	3/78							
1-51	Chev 3/4	78	12/78							
1-52	Chev 3/4	78	12/78							
1-53	Chev 3/4	78	12/78							
1-54	Chev 3/4	78	12/78							
1-55	Chev 3/4	78	12/78							
1-56	Chev 3/4	78	12/78							
1-57	Chev 3/4	78	12/78							
1-58	Dodge 1/2	79	3/79							
1-59	Dodge 1/2	79	3/79							

Water Systems Operation and Maintenance Workshop - 1990

REHABILITATION OF WATER CONVEYANCE SYSTEMS

Session 6-1

Will Samuel and Jerry Nypen

Rehabilitation and/or betterment of water system facilities may be needed for a variety of reasons which could range from minor modifications or repairs to a complete rehabilitation of the system. Reasons for rehabilitating a system may be general deterioration, water conservation, safety, increased water demands, operating efficiency, upgraded standards, to reduce O&M (Operation and Maintenance) costs, or environmental concerns.

If a rehabilitation program is contemplated, a survey of the system should be made to identify and prioritize, on a preliminary basis, the work to be undertaken and to establish objectives to be accomplished by the program.

The survey should be made by management and operations personnel of the operating organization and others who may be involved in the rehabilitation program (i.e., Bureau of Reclamation, state agencies, consulting firms, etc.)

A rehabilitation program may be funded with the annual O&M assessment of the water user entity or a special loan may be required. This would depend on the magnitude of the program and the repayment capability or willingness of the water users to repay.

If possible, the first method is the most desirable possibly as it will establish a solid and planned funding base for current and future O&M needs of the district. However, sometimes it is necessary to obtain a loan in order to fund and do the work in an optimum or reasonable length of time. Loans for this purpose are available, on projects constructed by the Bureau of Reclamation, under the R&B (Rehabilitation and Betterment) Program. A document entitled "Loans Under the Rehabilitation and Betterment Act of 1949" dated August 1988 gives details of this program. Copies will be available at the workshop. Loans obtained under the R&B Program are interest free for irrigation facilities and they must be repaid in the shortest time possible and no longer than 40 years.

Interest in this program should be expressed to the Bureau of Reclamation. See pages 13 and 14 of "The Rehabilitation and Betterment Program" for Regional Directors and jurisdiction boundaries.

To illustrate the possible advantages and benefits of undertaking a rehabilitation and/or betterment program, the following example is given. Benefits will obviously vary depending on specific conditions of a given water system.

Possible Benefits

- Keep system "whole" (reduce risks of failure)
- Improve deliveries/control (improve water service to users)
- Optimum O&M
- Water savings
- Less seepage/drainage (reclaim productive lands)
- Square off fields
- Safety
- Better field deliveries (gated pipe, sprinkler)
- Energy conservation

Concerns

- What is in it for us?
- Total costs up?
- Mitigation for wildlife
- Benefits vs costs

Actual R&B Project

Frenchman-Cambridge Irrigation District

Project

1. 45,000 acres - 190,000 - 200,000 acre-feet
2. 160 mi. canal - 112 mi. laterals
3. All ditches open
   - high seepage
   - high weed control costs
   - high cleaning costs
   - frequent repair
   - poor deliveries

Proposed Work

1. Replace 50 mi of ditches with pipe
2. Serve 13,000+ acres
3. 109 laterals and sublaterals
4. 2 - 12 cfs laterals replaced with 8 - 24 in. pipe
5. Construction with district people
6. 4-5 extra workers
7. Bought ditcher, dozer, dragline
8. Cost $4.4 million

Benefits

1. Less fluctuation --- better delivery-demand system
2. Safety around cities
3. No freezing problems
4. O&M costs
    - Before R&B $500/mi. (small laterals) weeds & maintenance
    - After R&B --- zilch! (0)
    - Before R&B - 1-1/2-2 men/canal for operations
    - After R&B - 1/2 man/canal
5. Pressure gated pipe
6. Better yields --- less water (2 yrs.)
7. Less seepage/drainage
8. Less total length
9. Better water measurement

"Everything expected and more!"

Proposed Rehabilitation

Project Data

Acreage	27,000+ acres
Diversion	140,000 acre-feet
Storage	Supply and diversion dam
Canal	750 cfs (max) - 46 mi.
Laterals	202 mi.
Drains	187 mi.
Cost	$7.8 million

Work

Canals	
- Lining	16 mi.
- Lateral T.O.	47 mi.
- Farm T.O.	150 mi.
Laterals	
- Lining	59.4 mi.
- Pipe	13.6 mi.
- T.O. (replace & mod.)	1168
- Flumes & Weirs	17
Drains	
- Open to closed	4000 ft

Repayment

1981 - 1984	$ -0-
1985 - 2020	212,000/yr
2021	168,000
	$7,800,000

Actions Needed to Implement

- determine need	district/Bureau
- resolution	district
- prepare report/review	Bureau
- contract negotiation	district/Bureau
- contract approval/submittal	district/Bureau
- Commissioner's approval, to Secretary	Commissioner
- 60 day review	committee
- district election (51%)	district
- court validation	court
- execute contract	Regional Director
- initiate work	district/contractor

After approval of the R&B Program, final designs should be prepared for the work to be done. The following information should be helpful in preparing designs for open water conveyance systems. See lecture notes for Session 8-1 for information on design, installation, and maintenance of pipe systems. Much of the material given in other sessions of this workshop is also pertinent to R&B work.

1. Open water conveyance canal sections
   a. Earth-lined section
   b. Concrete-lined section
   c. Buried membrane lined section

2. Open water conveyance system structures
   a. Check structures
   b. Inverted siphons
   c. Inline flume structures
   d. Pipe and rectangular drop and chute structures
   e. Turnout structures
   f. Wasteway structures
   g. Cross drainage structures
   h. Drainage inlets
   i. Bridges

See appendix A (page 14) for a listing of Bureau publications that may be helpful if a more thorough discussion is required on any of the above topics. There are blank pages provided at the end of the narrative. These may be used for questions that occur while reading this paper and also for note taking during the session. The session will be more beneficial and meaningful if the participants come prepared with questions and comments.

Open Water Conveyance Canal Sections

The majority of the older systems are either earth channels or earth-lined canals. There is a good possibility that earth canals that were not constructed with a compacted impervious earth lining are now experiencing seepage problems. In the past, seepage may not have been considered too great a problem, but with the increasing value of water and today's attitudes toward conservation, seepage has become a problem that should be given serious consideration. If seepage is in fact a serious problem and a solution is required to reduce the seepage, then lining may be required. The three most prevalent types of lining are earth lining, concrete lining, and buried membrane lining.

The rehabilitation may dictate that a different type of canal section will be required. For example, assume that the original canal was excavated in a pervious or semipervious material and that good earth lining material is not available, then a concrete or buried membrane section should be considered. Another example would be for a canal in a populated area where the canal has become "built in." A concrete section would be smaller; therefore, there would be more room for construction activities. And yet another example would be a new canal section required for an increased capacity. A concrete section may fit into the same right-of-way because the better Manning's "n" value for concrete would allow for a smaller section. Figures 1, 2, 3, and 4 are included to aid in selecting the proper cross section for a new canal section. Figure 5 shows the relationship between canal capacity, and lining freeboard and bank height for the various linings.

Earth-lined Section

If the proper earth materials are present in the project area, a compacted earth lining is one of the most effective ways of stopping seepage. The seepage rate for compacted earth lining, as well as for concrete lining, is 0.07 cubic foot per square foot per day. Figure 6 shows a listing of soils and their suitability for use as lining material and figure 7 shows typical earth-lined sections.

There may be cases where other methods of rehabilitating the earth canal section may be used. If a canal section is in a pervious sandy material, especially dune sand, then a mixture of this material and fly ash may produce a relatively watertight lining. Tests must be made to determine if there will be a favorable reaction to the mixture and also to determine the ratios of the fly ash to sand mix. There must also be a cheap supply of fly ash near the project.

If the canal embankment was built of an expansive clay and there has been stability problems, lime treatment may be the solution. The prism is overexcavated, to limits similar to that required for earth lining, and backfilled with a compacted earth-lime mixture. This mixture is soil with 3 to 5 percent lime by weight (tests are required to determine effectiveness and percentage of lime required).

Concrete-lined Section

The most common lining material in present use is concrete. The basic reasons for this are that modern trimming and lining jumbos have made concrete lining economical, that a concrete section is smaller, therefore less right-of-way is required; and that there is a lack of good earth lining material at many canal sites. Figures 8 and 9 show a typical concrete-lined section, while figure 10 shows the relationship between canal capacity and concrete lining thickness. The reinforced concrete lining, as shown in figure 9, should be considered when the canal crosses a deep gully or ravine, or when a siphon transition extends out onto high fill. The reinforced lining would prevent excessive seepage and a possible washout if cracking formed in the lining. In many cases a washout in these areas would be disasterous. Consideration should also be given to fully compacting the cross section in these high fill areas. This would reduce the possibility of differential settlement and would add some stability against erosion of the outer side slopes along the canal.

The contraction joints for unreinforced lining are shown in figure 11. Alternative No. 1, or a similar alternative, should be used for bulkheaded joints (used at the end of the day, when the lining machine breaks down, etc.). Care should be taken to assure a good fit at intersections.

Safety ladders for concrete-lined canals are shown in figure 12. The reason for the plastic sleeves and washers shown in detail A is to keep dissimilar metals and concrete out of contact with each other, therefore eliminating the possibility of an electrolysis problem.

Hydrostatic pressure behind the concrete lining may be encountered if, for any reason, the ground water builds up behind the lining. If this problem is anticipated, an underdrainage system with flap valve weeps may be used. Figure 13 shows a typical underdrain and figure 14 shows the flap valve weep.

Buried Membrane Lined Section

The Bureau of Reclamation has been doing research on the use of buried membrane lining. This research and past experience show that PVC (polyvinyl chloride) sheets appear to be the best material for membrane lining. In general, other materials such as asphalt have not held up well. Figure 15 shows a typical section for plastic lining. Another session in this workshop will go into more detail on plastic lining.

Open Water Conveyance System Structures

The function of open water conveyance system structures has not changed throughout the years, but changes have occurred in the hydraulic and/or structural designs of these structures. This paper will present the current thinking on the more common of these structures.

Check Structures

Check structures are generally used to regulate the flow in a canal and to provide the required head at upstream turnouts. Check structures provide an upstream-checked water surface, which is a level pool equal to the elevation of the design capacity normal depth at the check structure. The structures are usually spaced along the canal to maintain a 2- to 3-foot difference in checked water surfaces. It is economically advantageous to use check structures on inlets to inverted siphons, if a check is required near the siphon, since an inlet transition on the siphon is aleady required. Check structures are also generally required where there is a change in canal cross section.

In the past many of the check structures were built with stoplogs as the control. Stoplog check structures may still be desirable if the canal capacity is small and if very few changes are required during the operating season. If stoplog checks are used, the velocity through the stoplog section, based on design capacity, should be 3.5 feet per second or less. The stoplog grooves should have a 1/4:1 slope if the distance from the deck to the floor is greater than 6 feet. If normal depth is greater than 6 feet or if the opening is wider than 5 feet, then stoplog checks should not be used. Figure 16 shows a typical stoplog check structure.

The majority of the present check structures are designed with gates, the most common being the radial gate with cable hoists. Gated check structures are required if the canal is to be automated. Most gated structures are designed with overflow walls that would, in an emergency, bypass 20 to 30 percent of the design capacity before the lining upstream of the check is overtopped. It is desirable to have stoplog guides in the gate bay, upstream and downstream of the gate, to isolate the gate without having to evacuate the canal. Figure 17 shows a typical gated check structure.

Inverted Siphon

The inverted siphon, which the Bureau refers to as a siphon, is used to convey the canal water under roads, streams, or other obstacles. The siphon shapes used have included box, trapezoidal, and pipe. Many older siphons tend to lose water, usually through bad joints.

The pipe siphon is the most common because, in most cases, it is the most economical. Also, a pipe siphon should be used for heads greater than 30 feet. For pipes up to 96 inches in diameter with heads up to 125 feet, standard precast reinforced concrete pressure pipe may be used. For diameters over 96 inches, either monolithic concrete pipe or precast pipe can be used. Pipes up to 21 feet in diameter have been precast on the jobsite for Bureau projects. When the head on the siphon exceeds 125 feet, then monolithic pipe with a steel liner, precast concrete pipe with a steel cylinder, pretensioned concrete pipe, or prestressed concrete pipe may be used. See figure 18 for a typical pipe siphon. For an example of "computing hydraulic losses through a pipe siphon," see figure 19. The procedure shown in figure 19 can be used for any shaped siphon.

If it becomes impractical to dewater a siphon from the transitions, a blowoff structure should be constructed at the low point of the siphon barrel. A manhole, for access to the siphon, may be included in the blowoff structure. If the siphon is exceptionally long, additional manhole structures may be desirable. For a typical blowoff structure see figure 20.

The following criteria should be considered when designing siphons because of modern construction equipment and techniques. The criteria are for monolithic pipe but portions of the criteria may be used for precast pipe. Check to determine if a direct wheel load (H20-44) can be taken on the pipe (monolithic only). The forms may not be removed until 25 percent of the specified 28-day compressive concrete strength is reached (monolithic only). Backfill may be placed below the spring line when the concrete has reached 50 percent of the specified 28-day compressive concrete strength or a period of 7 days has elapsed (monolithic only). Backfill may not be placed above the spring line and wheel loads may not be placed on the pipe until the specified 28-day compressive concrete strength is reached (monolithic only). Backfill will be brought up equally on both sides of the pipe. Heavy equipment crossings will not be allowed unless the backfill is compacted above the pipe to a distance that will give the cover required to safely spread the wheel loads over the pipe so as to not exceed its cover class. Heavy equipment shall be defined as that greater than H20-44 highway loading. There will be no stockpiling of any material over the siphon pipe that would increase the equivalent earth load to greater than indicated by the class of the pipe.

The box siphon is usually used when the siphon has a low head, 30 feet or less to the top of the box, and is relatively short. A box siphon may be economical if the required pipe size is not readily available in the area or if no other pipe is required in the contract. The trapezoidal siphon may be used when there is a short road crossing such as a county road. A trapezoidal siphon may also be used under a streambed as shown in figure 21. In this case it is called a trapezoidal wash siphon. The trapezoidal section may be more economical than a pipe section for a short siphon because the trapezoidal section does not have any transitions. Cost curves can be made to determine at what point pipe becomes more economical than the trapezoidal section. The trapezoidal siphon may also be advantageous where head is a problem because losses through a trapezoidal siphon are minimal.

Inline Flume Structures

Flume structures can be used to convey water along a steep hillside or over a minor depression. The flume on a hillside is called a bench flume and a flume over a depression is called an elevated flume. Elevated steel flumes with a semicircular shape and elevated reinforced concrete flumes were the most prevalent flumes constructed in the past.

When an elevated flume has deteriorated beyond the point of repair, a pipe siphon should be considered for replacement because of maintenance problems, vandalism problems, and environmental considerations. Also, a pipe siphon will generally be more economical than an elevated flume.

Bench flumes are used for construction along a steep hillside. If there is a possibility of a large amount of falling debris or if the capacity is small, then a pipeline for bench construction may be used. Figure 22 shows a typical bench flume and pipeline for flume construction.

Drop and Chute Structures

Drop structures are used to convey water to a lower elevation, with a maximum change in water surface of 15 feet, while a chute is used to convey water over longer distances and through greater changes in grade. Pipe drops may be used for smaller capacity systems or where the water must also be taken under a road. The type 1 pipe drops, shown in figure 23, should be used only when the possibility of clogging is minimal. If sediment or debris is carried by the canal water, a pipe drop similar to the type 2 pipe drop shown in figure 24 should be used. Special consideration should be given to the type of outlet used on the type 2 pipe drop if there is a large amount of debris, especially large tumbleweeds. Tumbleweeds mixed with sediment have completely clogged the baffled-type outlet. For larger capacity systems, a rectangular drop with a stilling basin, such as shown in figure 25, should be used. The rectangular drop structures usually have a floor slope of 2:1 and are used for a maximum water surface drop of 15 feet.

The pipe and rectangular drops should have a check, a weir, stoplogs, or a control notch on the inlet to control the upstream water surface for a range of flow capacities.

A baffled apron drop may be used as an alternate drop structure. A stilling basin is not required and there are no tailwater requirements with a baffled apron drop. Also, there are no head limitations on a baffled apron drop. The baffled apron drop is ideal for a terminal structure where the tailwater fluctuates. Figure 26 shows a typical baffled apron drop.

Vertical drops of 3 feet or less can be made with, and are usually incorporated into, a check structure. A typical check drop structure is shown in figure 27.

Pipe chutes may have a stilling basin outlet or a baffled outlet as shown in figure 28. The pipe chutes are usually used for smaller capacities, but also allow for crossings or for other use of the land over the pipe. For larger capacities, a rectangular chute with a stilling basin, similar to that shown in figure 28, should be used. As with drop structures, some type of control structure should be put on the inlet of all chute structures.

Turnout Structures

In the past, the majority of turnouts were designed to deliver water to open gravity irrigation systems. Two of the more common turnouts for this purpose, the constant-head orifice turnout and the open flowmeter turnout, are shown in figure 29. A trashrack may be desirable on the turnout with the open flowmeter to keep trash out of the propeller. A turnout with a weir box, as shown in figure 30, may be used for 5 cubic feet per second or less. Parshall flumes or weir structures, as shown in figure 31, may be used in conjunction with a gated turnout by placing these structures below the turnout. They should be placed far enough below the turnout to assure quiet water and a uniform velocity approach. The Parshall flume is generally maintenance-free and is not as sensitive to backwater as is the weir structure. The area upstream of weir structures should be kept free of weeds and silt for proper operation. In all cases, the turnouts are controlled by gates on the turnout headwall.

In recent years, sprinkler irrigation has gained in popularity. If turnouts are to serve sprinkler systems, the water should be screened before the water reaches the sprinkler system. This can be accomplished by placing removable screens with stationary guides on the turnout structure or by providing self-cleaning traveling water screens just downstream of the turnout structure. If the water is to be pumped, it may be measured downstream of the pumps by a Venturi meter. If the turnout serves a combination of gravity and pumped flow, the flow can be measured by a Venturi meter or an open flowmeter before the flow is split. Figure 32 shows two typical turnouts that may be used for a sprinkler irrigation system.

Wasteways

If a canal system that is being rehabilitated was initially constructed with few or no wasteways, then wasteways should be considered in the rehabilitation program, especially if the new section is concrete lined. The three common types of wasteways in use are the gated wasteway, the side channel wasteway, and the siphon wasteway. The advantage of the side channel wasteway, shown in figure 33, or the siphon wasteway, shown in figure 34, is that they will automatically discharge excess flow in the canal without any automation or power requirements. A slide gate may be added to a side channel wasteway or a siphon wasteway for dewatering the canal. The side channel wasteway will skim off small excesses and will slowly increase the waste capacity until the design discharge is reached, while the siphon wasteway must build up to prime the siphon, which results in a large, sudden discharge. If the design discharge is large and there is any danger of persons being in the downstream waste channel, an alarm should be put on the siphon wasteway.

The gated wasteway, shown in figure 35, has the advantage of being able to evacuate the canal. Slide gates or radial gates may be used. An advantage of radial gates is that they can be fully opened to pass floating debris. A disadvantage of the gated wasteway is that it either has to be operated manually or it must have power and automatic controls to operate when excess water must be passed.

Cross Drainage Structures

One of the most important things to consider for a canal rehabilitation is the condition and adequacy of the existing cross drainage structures. If overchutes tend to overflow, they can wash out the uphill canal bank. If the culverts do not have watertight joints, then piping of embankment material can take place with the possibility of a canal embankment failure.

Overchutes are used to convey drainage water over the canal in areas where the canal is in cut. Pipe overchutes, as shown in figure 36, can be used for smaller capacities, but should not be used if there is any danger of plugging with trash. Flume overchutes, as shown in figure 37, may be used for larger capacities or for water carrying a sizable amount of debris. Pipe overchutes may be sized based on a velocity of up to 10 feet per second using the 25-year storm and full pipe flow. The system should be checked using the 100-year storm to insure the upstream bank is not overtopped. If it is, then the upstream bank should be raised to contain the 100-year storm. A flume overchute should have adequate freeboard to contain the 100-year storm. For very large capacity drainages, with the original ground surface level around the same elevation as the normal water surface in the canal, a trapezoidal wash siphon may be used. Figure 21 shows a typical trapezoidal wash siphon. In some cases an alternative to an overchute would be a pipe culvert with a "bathtub" sediment basin at the outlet. The sediment basin should be designed to hold 1 year's worth or some other predetermined amount of sediment. This type of culvert is shown in figure 38.

Box culverts are generally used when the flows are very large or when the amount of fill in the culvert would exclude the use of pipe. The box section of the culvert should have waterstop joints every 25 feet. Collars should be used to prevent the possibility of piping along the box section. The cross section should be sized based on a full-section velocity of 10 feet per second for the 25-year storm. The culvert should be checked to see if the 100-year storm can be passed without overtopping the canal embankment.

Pipe culverts, as shown in figure 38, are the most common culverts when the fill is less than 20 feet because precast pipe is more economical than monolithic box culverts. The pipe culverts should be sized in the same manner as the box culverts are sized. The culverts should be constructed with a precast pipe that has watertight rubber gasket joints. Pipe collars should be used to prevent piping along the pipe. There should be a flexible pipe joint a distance of one-half of a pipe diameter, or 18 inches minimum, from the inlet transition, from the outlet transition, and from each side of the pipe collar, to prevent cracking the pipe.

Drainage Inlets

Drainage inlets are quite common in older canals. Because of water quality problems and the silt that can be introduced into the canal, very little drainage water should be allowed into the rehabilitated canals. This is especially true if the canal is to be concrete lined. The drainage water that could be allowed into the canal would be water from the O&M roads when the canal is in cut and small isolated drainage areas. The total amount of drainage water allowed into the canal should never exceed 10 percent of the canal design capacity. A CMP (corrugated metal pipe) under the O&M road or a concrete slab dropping the O&M road to the level of the top of the lining may be used to carry the water into the canal.

Canal Bridges

The types of bridges that will generally be encountered during a rehabilitation program will be short-span timber or cast-in-place concrete bridges, and an occasional steel-beam bridge. In most cases these bridges do not meet current loading standards and bridge width requirements. Therefore, factors to consider for rehabilitation are load capacity, vehicle or farm equipment widths, traffic volume, and State or county standards.

Current practice in Bureau bridge design is to provide single-span precast, prestressed concrete bridges when feasible. Precast, prestressed I-beams with a cast-in-place deck, as shown in figure 39, are used for spans of 50 to 100 feet, while precast, prestressed concrete, voided-slab or box units, as shown in figure 40, are used for shorter spans. The voided-slab or box unit bridges may require an asphalt or concrete wearing surface. Since not all precast sections are available in all locations, the availability of a desired section should be determined prior to design.

The bridge widths of county road or State highway bridges are determined by the county or State requirements. Farm bridge widths are usually determined by the size of the farm equipment used in the area. In some cases, farm bridges are designed with removable guardrails on one side to facilitate the movement of wide equipment. Designs are usually based on an HS20 truck, but never less than an H20 truck.

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APPENDIX A

Bureau of Reclamation publications that may be useful in water conveyance rehabilitation programs:

Design Standards No. 3 - Canals and Related Structures

Design of Small Canal Structures

Earth Manual - Second Edition

Concrete Manual - Eighth Edition

Water Measurement Manual

Hydraulic and Excavation Tables

Engineering Monograph No. 25 - Hydraulic Design of Stilling Basins and Energy Dissipators

Engineering Monograph No. 33 - Hydraulic Design of Transitions for Small Canals

Design Criteria for Concrete Retaining Walls

Linings for Irrigation Canals

Economic Justification for Canal Lining in Irrigation Distribution Systems

Design of Small Dams - Second Edition

Ground Water Manual

Standard Specifications for Reinforced Concrete Pressure Pipe

Soils as an Engineering Material - Report No. 17

Hydraulic Design of Stilling Basin for Pipe or Channel Outlets - Report No. 24

Slug Flow in Steep Chutes - Report CB-2

Hydraulic Model Studies of Corrugated Metal Pipe Underdrain Energy Dissipators - REC-ERC-71-10

Reducing Hazards to People and Animals on Reclamation Canals - REC-ERC-71-36

Design Standards No. 1 - Chapter 3, Safety Design Standards

Questions and Notes

Questions and Notes

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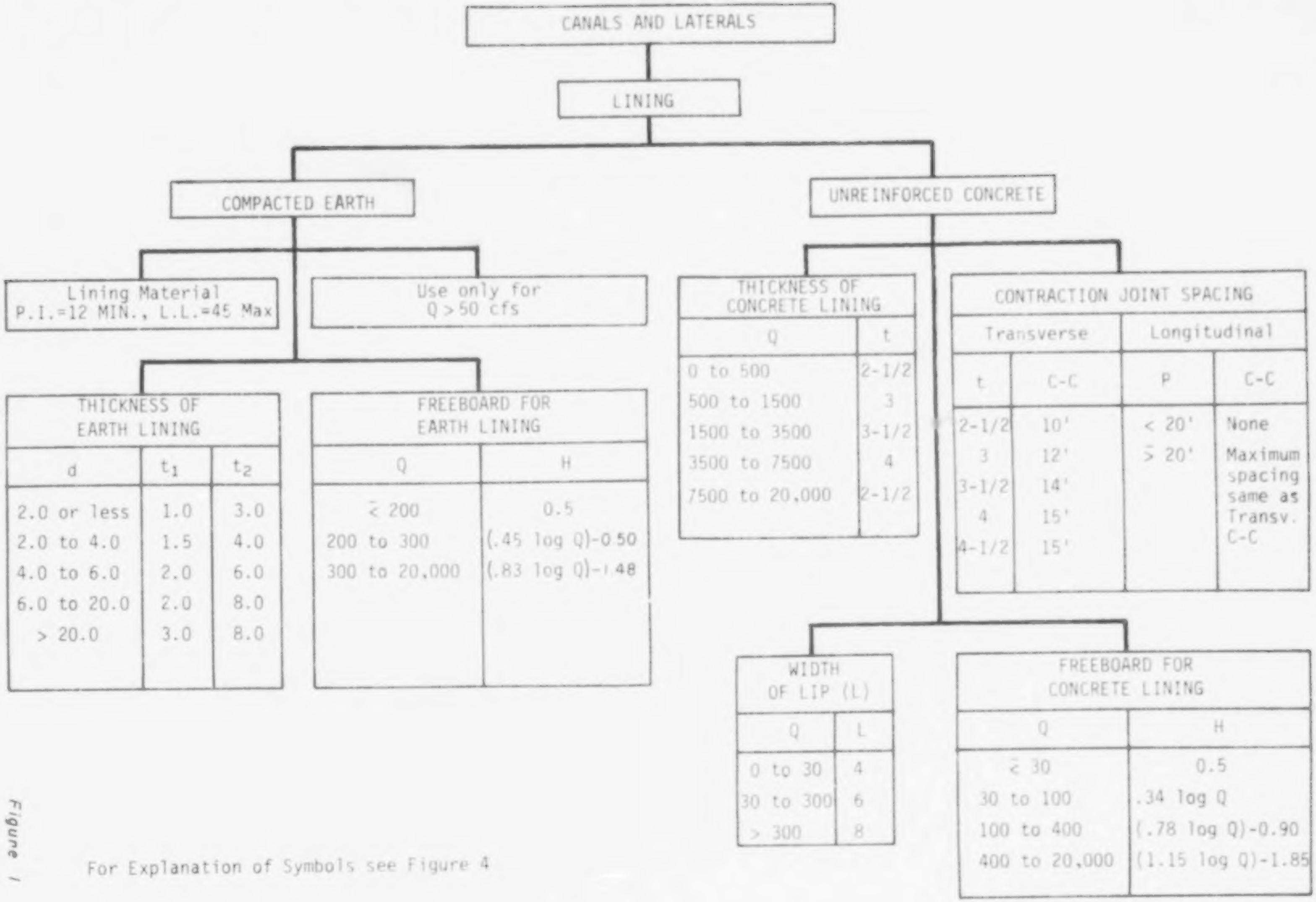


Lining Material
P.I.=12 MIN., L.L.=45 Max

Use only for
Q > 50 cfs

THICKNESS OF EARTH LINING

d	t_1	t_2
2.0 or less	1.0	3.0
2.0 to 4.0	1.5	4.0
4.0 to 6.0	2.0	6.0
6.0 to 20.0	2.0	8.0
> 20.0	3.0	8.0

FREEBOARD FOR EARTH LINING

Q	H
≤ 200	0.5
200 to 300	(.45 log Q)-0.50
300 to 20,000	(.83 log Q)-1.48

THICKNESS OF CONCRETE LINING

Q	t
0 to 500	2-1/2
500 to 1500	3
1500 to 3500	3-1/2
3500 to 7500	4
7500 to 20,000	2-1/2

CONTRACTION JOINT SPACING

Transverse		Longitudinal	
t	C-C	P	C-C
2-1/2	10'	< 20'	None
3	12'	≥ 20'	Maximum spacing same as Transv. C-C
3-1/2	14'		
4	15'		
4-1/2	15'		

WIDTH OF LIP (L)

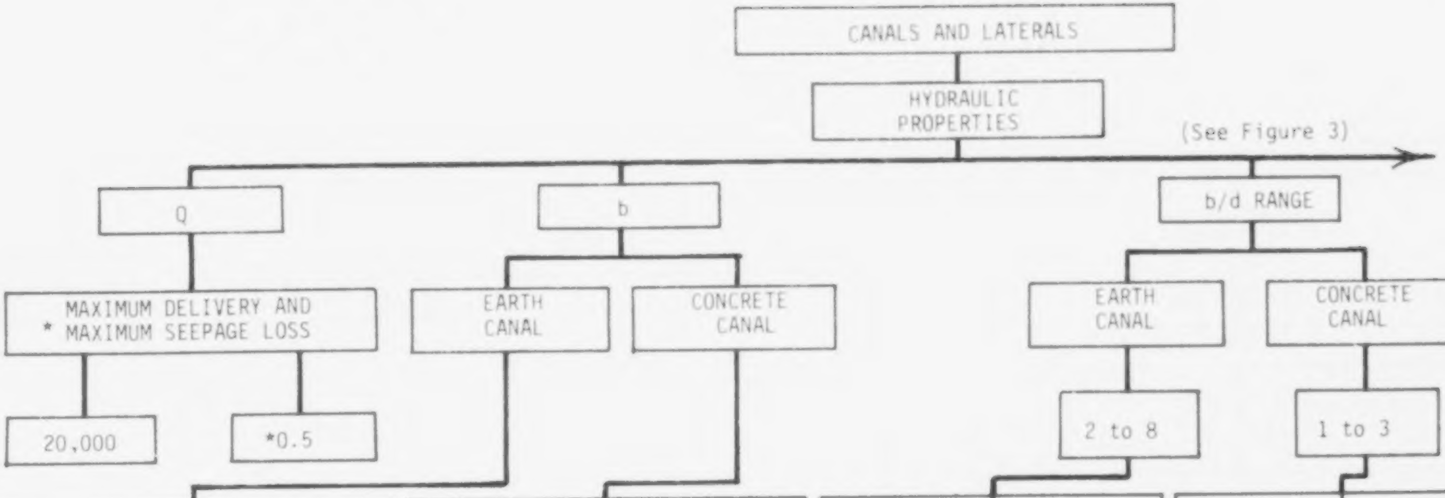
Q	L
0 to 30	4
30 to 300	6
> 300	8

FREEBOARD FOR CONCRETE LINING

Q	H
≤ 30	0.5
30 to 100	.34 log Q
100 to 400	(.78 log Q)-0.90
400 to 20,000	(1.15 log Q)-1.85

For Explanation of Symbols see Figure 4

Figure 1


CANALS AND LATERALS
HYDRAULIC PROPERTIES
(See Figure 3)
Q
b
b/d RANGE
MAXIMUM DELIVERY AND
* MAXIMUM SEEPAGE LOSS
EARTH CANAL
CONCRETE CANAL
EARTH CANAL
CONCRETE CANAL
20,000
*0.5
2 to 8
1 to 3

Q	b
50 to 100	6.64(log Q)-5.3
100 to 250	10.1(log Q)-12.2
250 to 1000	23.3(log Q)-43.9
1000 to 2500	47.7(log Q)-117.1
2500 to 5000	66.4(log Q)-180.8
5000 to 7500	114(log Q)-357
7500 to 10,000	120(log Q)-380
10,000 to 15,000	199(log Q)-696
15,000 to 20,000	280(log Q)-1035

*Maximum allowable seepage loss before lining is required (cfs/sq. ft/day)

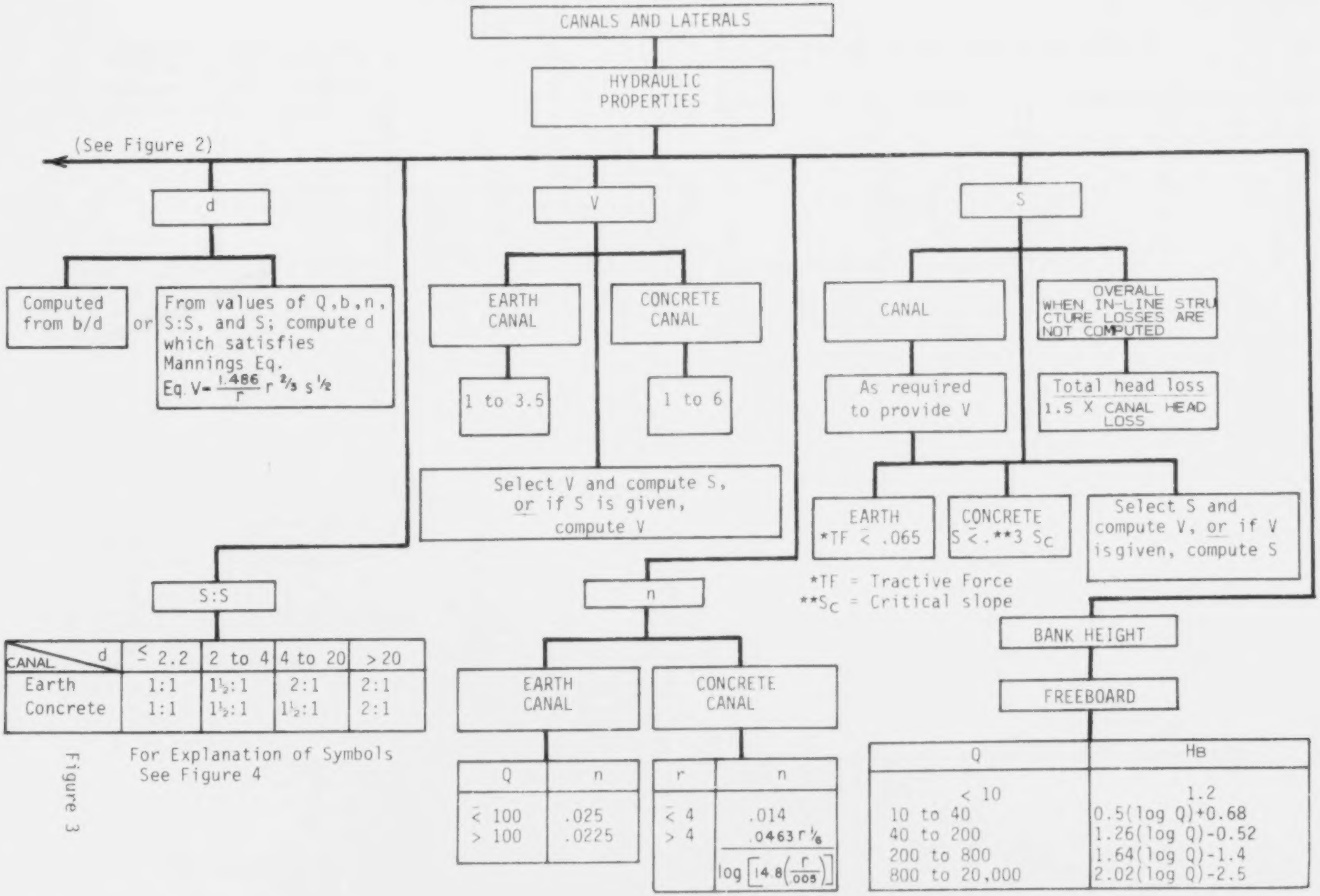
Q	b
≤ 4	1.0
4 to 15	2.0
15 to 30	3.0
30 to 60	4.0
60 to 100	5.0
100 to 250	6.0
250 to 800	3.96(log Q)-3.5
800 to 2500	14.1(log Q)-32.9
2500 to 4500	39.2(log Q)-118.3
4500 to 7500	67.6(log Q)-222
7500 to 10,000	80(log Q)-270
10,000 to 15,000	142(log Q)-518
15,000 to 20,000	200(log Q)-760

Q	b/d Minimum
50 to 100	2
100 to 500	2.5
500 to 1000	3
1000 to 2500	3.5
2500 to 5000	4
5000 to 7500	5
7500 to 10,000	6
10000 to 15000	7
15000 to 20000	8

Q	b/d Minimum
0.5 to 25	1.0
25 to 500	1.2
500 to 2500	1.3
2500 to 5000	1.5
5000 to 7500	1.8
7500 to 10,000	2.0
10000 to 15000	2.5
15000 to 20000	3.0

For Explanation of Symbols see Figure 4

Figure 2


CANALS AND LATERALS
HYDRAULIC PROPERTIES
(See Figure 2)
d
Computed from b/d
or
From values of Q,b,n, S:S, and S; compute d which satisfies Mannings Eq.
Eq V= $\frac{1.486}{r} r^{2/3} s^{1/2}$
V
EARTH CANAL
1 to 3.5
CONCRETE CANAL
1 to 6
Select V and compute S, or if S is given, compute V
S
CANAL
As required to provide V
EARTH
*TF ≤ .065
CONCRETE
S ≤ .**3 S_c
Select S and compute V, or if V is given, compute S
OVERALL WHEN IN-LINE STRUCTURE LOSSES ARE NOT COMPUTED
Total head loss
1.5 X CANAL HEAD LOSS
S:S
n
EARTH CANAL
CONCRETE CANAL
BANK HEIGHT
FREEBOARD

CANAL \ d	≤ 2.2	2 to 4	4 to 20	> 20
Earth	1:1	1½:1	2:1	2:1
Concrete	1:1	1½:1	1½:1	2:1

For Explanation of Symbols
See Figure 4

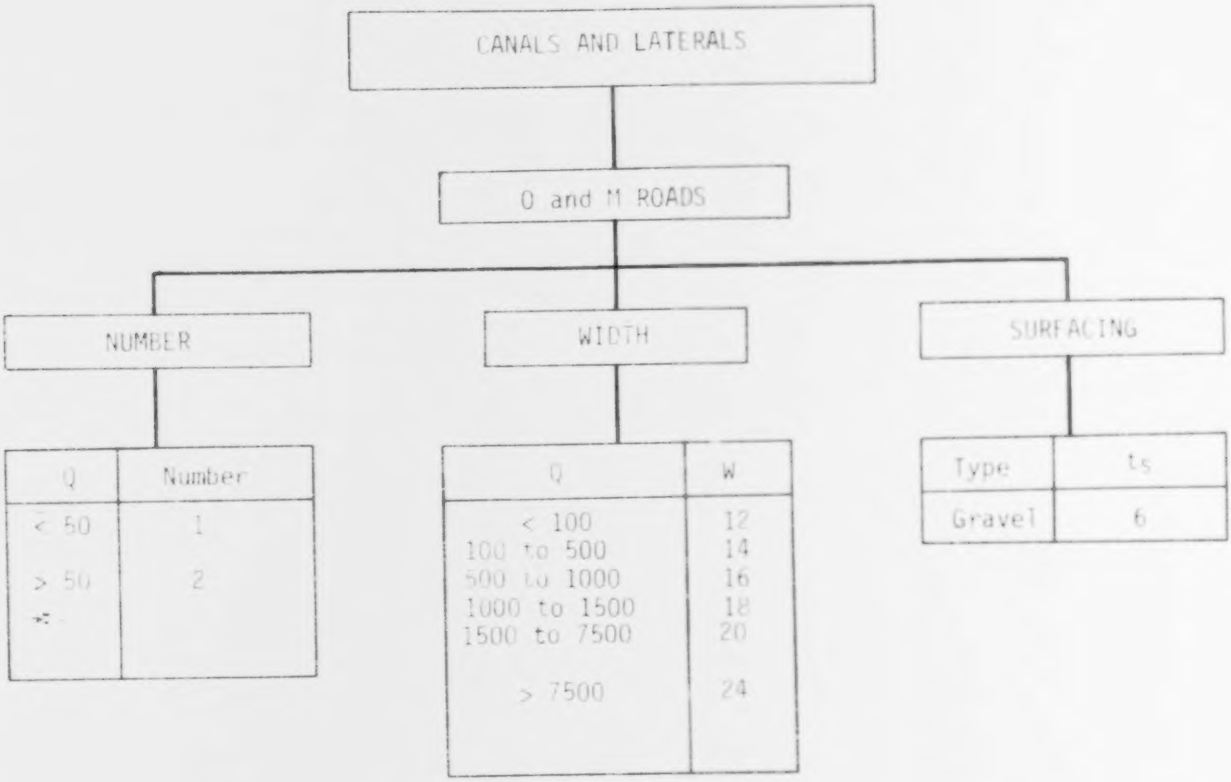
Q	n
≤ 100	.025
> 100	.0225

r	n
≤ 4	.014
> 4	$\frac{.0463 r^{1/6}}{\log\left[14.8\left(\frac{r}{.005}\right)\right]}$

*TF = Tractive Force
**S_c = Critical slope

Q	H_B
< 10	1.2
10 to 40	0.5(log Q)+0.68
40 to 200	1.26(log Q)-0.52
200 to 800	1.64(log Q)-1.4
800 to 20,000	2.02(log Q)-2.5

Figure 3



Q	Number
< 50	1
> 50	2

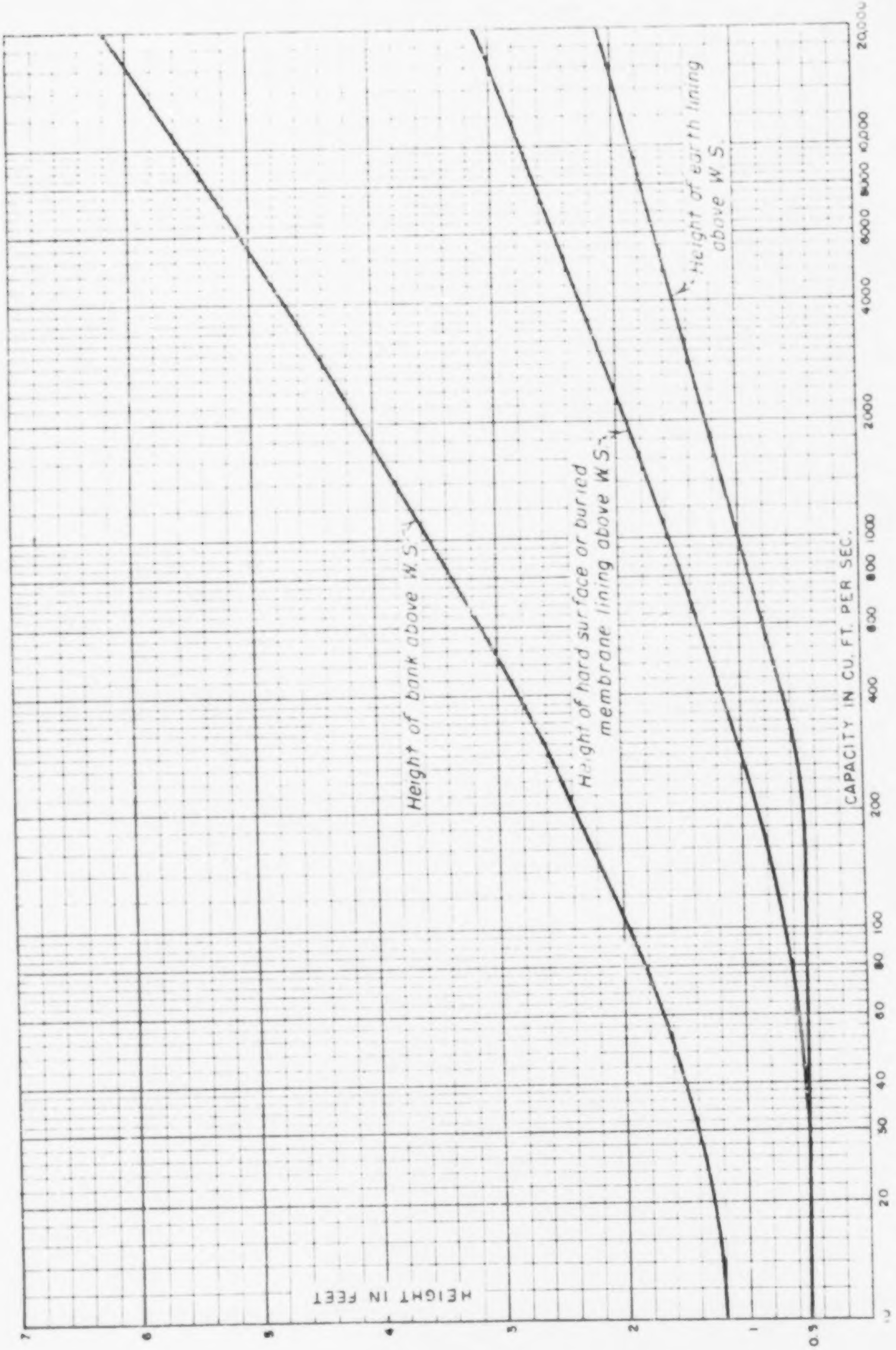
Q	W
< 100	12
100 to 500	14
500 to 1000	16
1000 to 1500	18
1500 to 7500	20
> 7500	24

Type	t_S
Gravel	6

Explanation of Symbols:

d = Normal depth (ft)
NWS = Normal water surface
t = Thickness of concrete lining (inches)
t_1 = Vertical thickness of earth lining at invert (ft)
t_2 = Horizontal thickness of earth lining along sideslopes (ft)
b = Canal bottom width (ft)
Q = Canal design capacity (cfs)
V = Canal velocity (ft/sec)
r = Hydraulic radius
n = Mannings "n"
S = Energy slope
S_C = Critical energy slope
S:S = Canal sideslopes
H_E = Height of earth lining above NWS (ft)
H_C = Height of concrete lining above NWS (ft)
H_B = Height of bank above NWS (ft)
L = Width of lining lip (ft)
P = Perimeter of concrete lining (ft)
C-C = Center to center spacing of contraction joints
TF = Tractive force
W = Width of O and M Road (ft)
t_S = Thickness of gravel surfacing (inches)

Figure 4



FREEBOARD AND BANK HEIGHT FOR CANAL SECTIONS

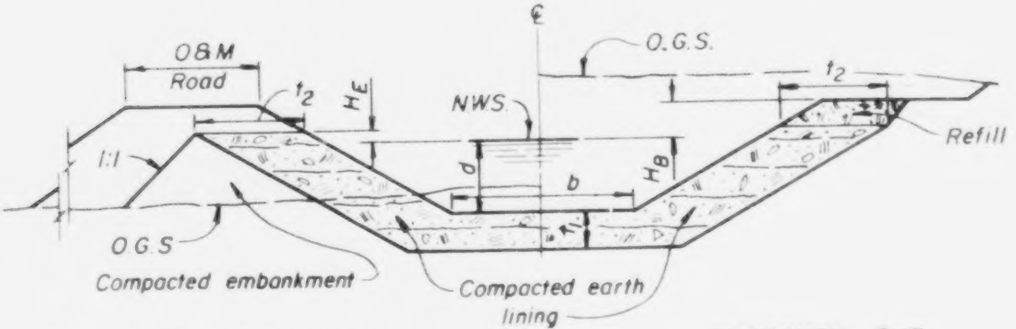
MAJOR DIVISIONS OF SOILS			TYPICAL NAMES OF SOIL GROUPS	GROUP SYMBOLS	SOIL PROPERTIES			SUITABILITY FOR CANALS	
					PERMEABILITY	SHEARING STRENGTH	COMPACTED DENSITY	EROSION RESISTANCE	COMPACTED EARTH LININGS
COARSE GRAINED SOILS More than half of material is larger than No 200 sieve size (The No 200 sieve size is about the smallest particle visible to the naked eye)	GRAVELS More than half of coarse fraction is larger than No 4 sieve size (For visual classifications, the ¼ size may be used as equivalent to the No 4 sieve size)	CLEAN GRAVELS (Little or no fines)	Well-graded gravels, gravel-sand mixtures, little or no fines	GW	14	16	15	2	—
			Poorly graded gravels, gravel-sand mixtures, little or no fines	GP	16	14	8	3	—
		GRAVELS WITH FINES (Appreciable amount of fines)	Silty gravels, poorly graded gravel-sand-silt mixtures	GM	12	10	12	5	6
			Clayey gravels, poorly graded gravel-sand-clay mixtures	GC	6	8	11	4	2
			Gravel with sand-clay binder	GW-GC	8	13	16	1	1
	SANDS More than half of coarse fraction is smaller than No 4 sieve size	CLEAN SANDS (Little or no fines)	Well-graded sands, gravelly sands, little or no fines	SW	13	15	13	8	—
			Poorly graded sands, gravelly sands, little or no fines	SP	15	11	7	9 coarse	—
		SANDS WITH FINES (Appreciable amount of fines)	Silty sands, poorly graded sand-silt mixtures	SM	11	9	10	10 coarse	7 Erosion Critical
			Clayey sands, poorly graded sand-clay mixtures	SC	5	7	9	7	4
			Sand with clay binder	SW-SC	7	12	14	6	3
FINE GRAINED SOILS More than half of material is smaller than No 200 sieve size	SILTS AND CLAYS Liquid limit less than 50		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	ML	10	5	5	—	8 Erosion Critical
			Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	CL	3	6	6	11	5
			Organic silts and organic silt-clays of low plasticity	OL	4	2	3	—	9 Erosion Critical
	SILTS AND CLAYS Liquid limit greater than 50		Inorganic silt, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	9	3	2	—	—
			Inorganic clays of high plasticity, fat clays	CH	1	4	4	12	10, Volume Change Critical
			Organic clays of medium to high plasticity	OH	2	1	1	—	—
HIGHLY ORGANIC SOILS			Peat and other highly organic soils	Pt	*			**	

* Numbers above indicate the order of increasing values for the physical property named

** Numbers above indicate relative suitability (1=best)

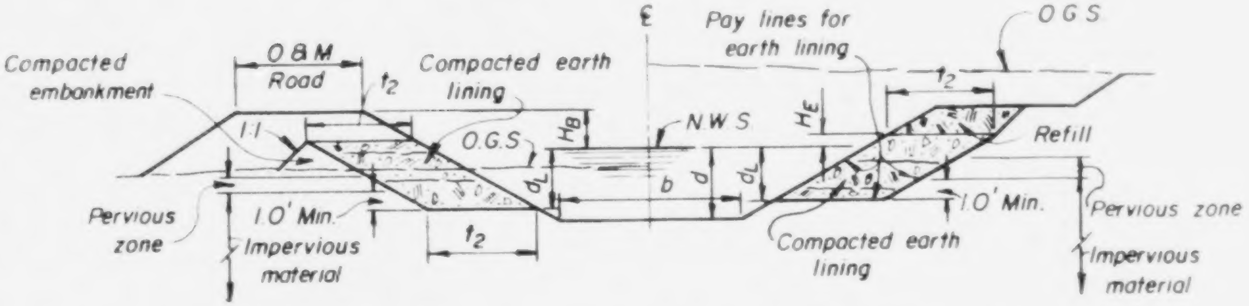
SOIL SUITABILITY FOR EARTH LINING

Figure 6


℄
O.G.S.
O&M Road
t_2
H_E
N.W.S.
t_2
Refill
1:1
d
b
H_B
t_1
O.G.S
Compacted embankment
Compacted earth lining

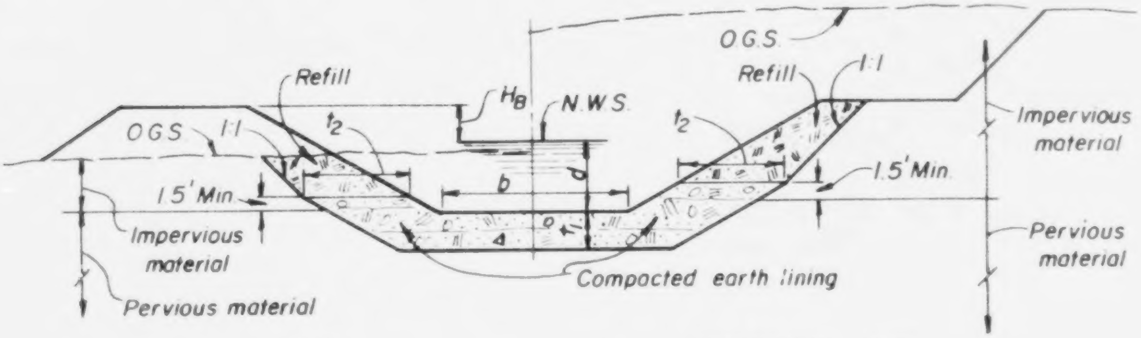
PARTIAL FILL THOROUGH CUT

TYPE I EARTH LINING
(BOTTOM AND SIDES)


℄
Pay lines for earth lining
O.G.S
Compacted embankment
O&M Road
t_2
Compacted earth lining
t_2
1:1
H_B
O.G.S
N.W.S
H_E
Refill
d_L
b
d
d_L
Pervious zone
1.0' Min.
Impervious material
t_2
1.0' Min.
Pervious zone
Compacted earth lining
Impervious material

PARTIAL FILL THOROUGH CUT

TYPE 2 EARTH LINING
(SIDES ONLY)


O.G.S.
Refill
1:1
Refill
H_B
N.W.S.
O.G.S
1:1
t_2
t_2
Impervious material
d
b
1.5' Min.
1.5' Min.
t_1
Impervious material
Pervious material
Compacted earth lining
Pervious material

PARTIAL FILL THOROUGH CUT

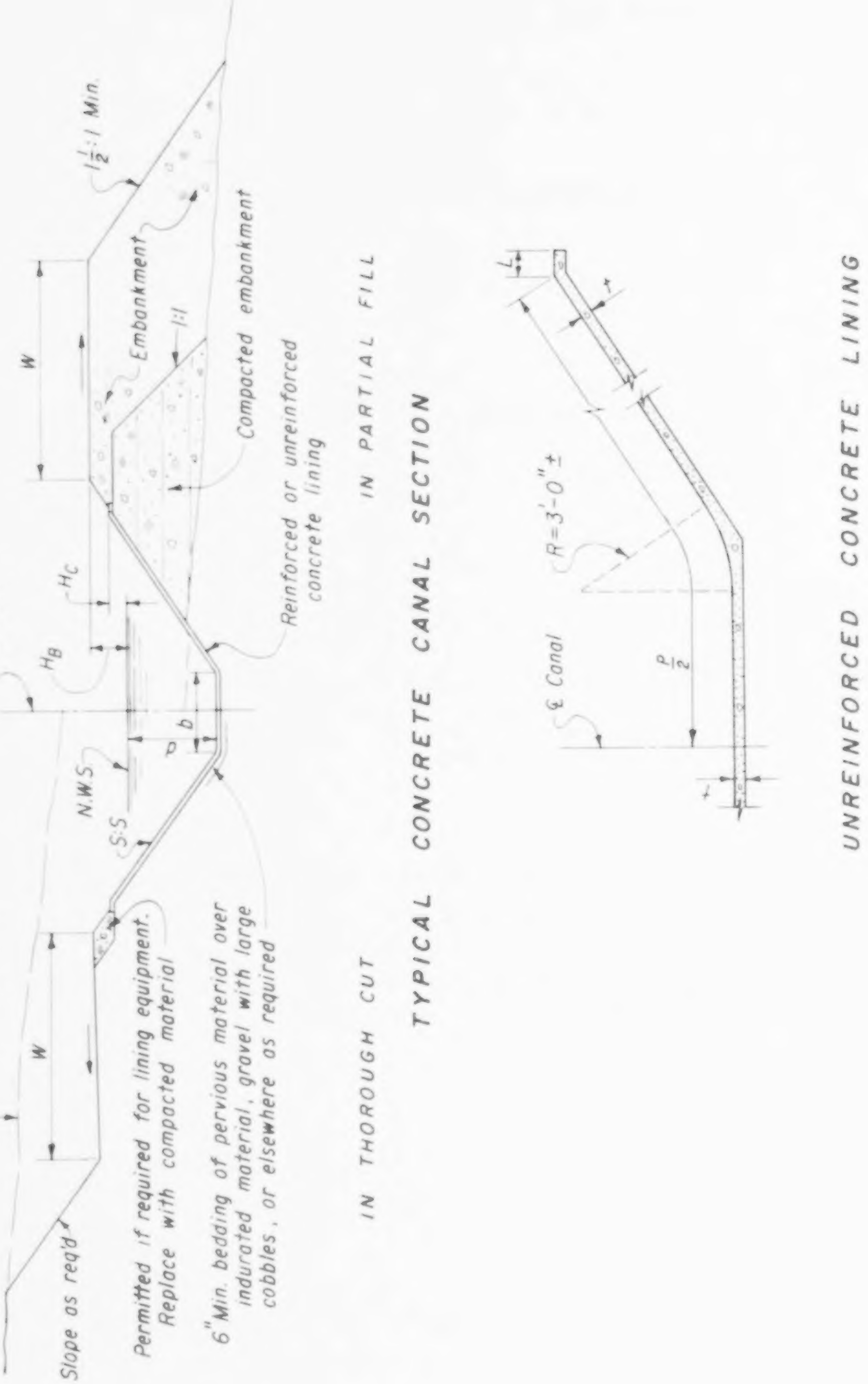
TYPE 3 EARTH LINING
(BOTTOM ONLY)

NOTES

If lining material requires a protective cover of gravel or riprap to prevent scour or erosion, excavation pay lines shall be extended to provide for the designated thickness of lining plus the gravel or riprap cover.

If a pervious zone containing mostly course material is encountered and requires a graded filter to prevent loss of lining material, excavation pay lines shall be lowered on sides or bottom to provide for the designated thickness of lining plus the filter.

EARTH LINING SECTIONS

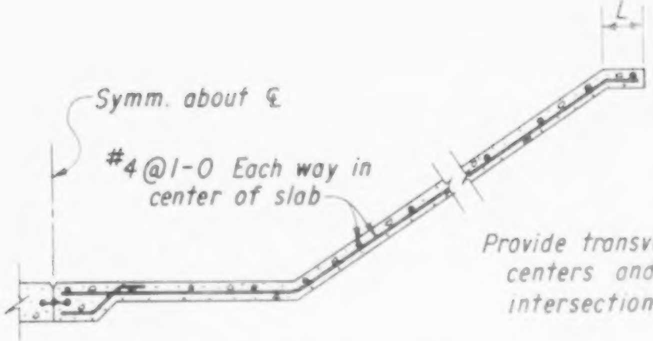

Original ground surface
Slope as req'd
W
Permitted if required for lining equipment. Replace with compacted material
6" Min. bedding of pervious material over indurated material, gravel with large cobbles, or elsewhere as required
S.S
N.W.S.
d
b
℄ Canal
H_B
H_C
Reinforced or unreinforced concrete lining
Compacted embankment
Embankment
1:1
W
$1\frac{1}{2}$:1 Min.
IN THOROUGH CUT
IN PARTIAL FILL
℄ Canal
$\frac{P}{2}$
R=3'-0"±
t
L

TYPICAL CONCRETE CANAL SECTION

UNREINFORCED CONCRETE LINING

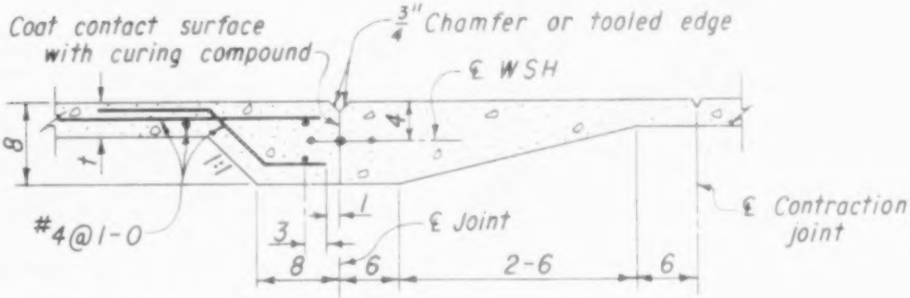
CONCRETE LINING SECTION - SHEET I

Figure 8

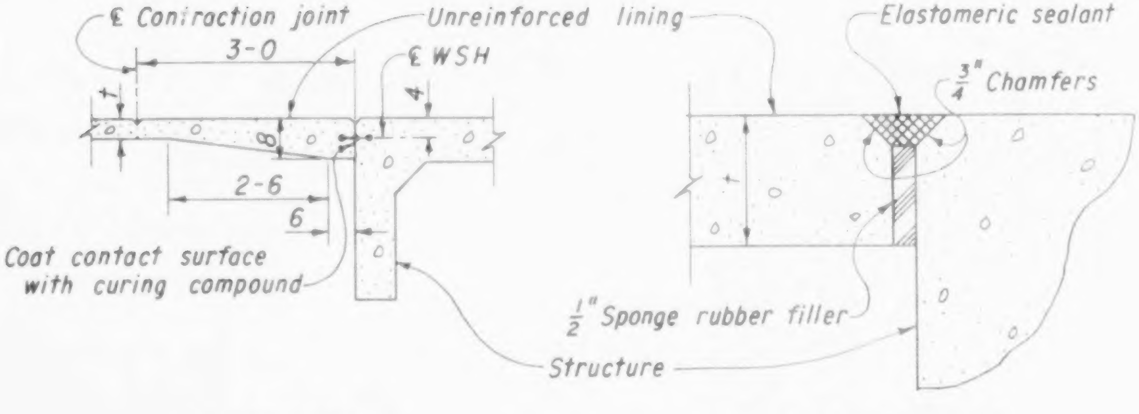

L
Symm. about ℄
#4@1-0 Each way in center of slab

Provide transverse joints at 25'-0" centers and vulcanize waterstop intersections as required

REINFORCED CONCRETE LINING


Coat contact surface with curing compound
$\frac{3}{4}$" Chamfer or tooled edge
℄ WSH
8
4
1
1:1
#4@1-0
3
1
℄ Joint
8
6
2-6
6
℄ Contraction joint

WATERSTOP JOINTS IN LINING
(Reinforced to unreinforced lining shown. Symm. about ℄ Joint for joint in reinforced lining.)


℄ Contraction joint
3-0
Unreinforced lining
℄ WSH
4
1
8
2-6
6
Coat contact surface with curing compound
Elastomeric sealant
$\frac{3}{4}$" Chamfers
t
$\frac{1}{2}$" Sponge rubber filler
Structure

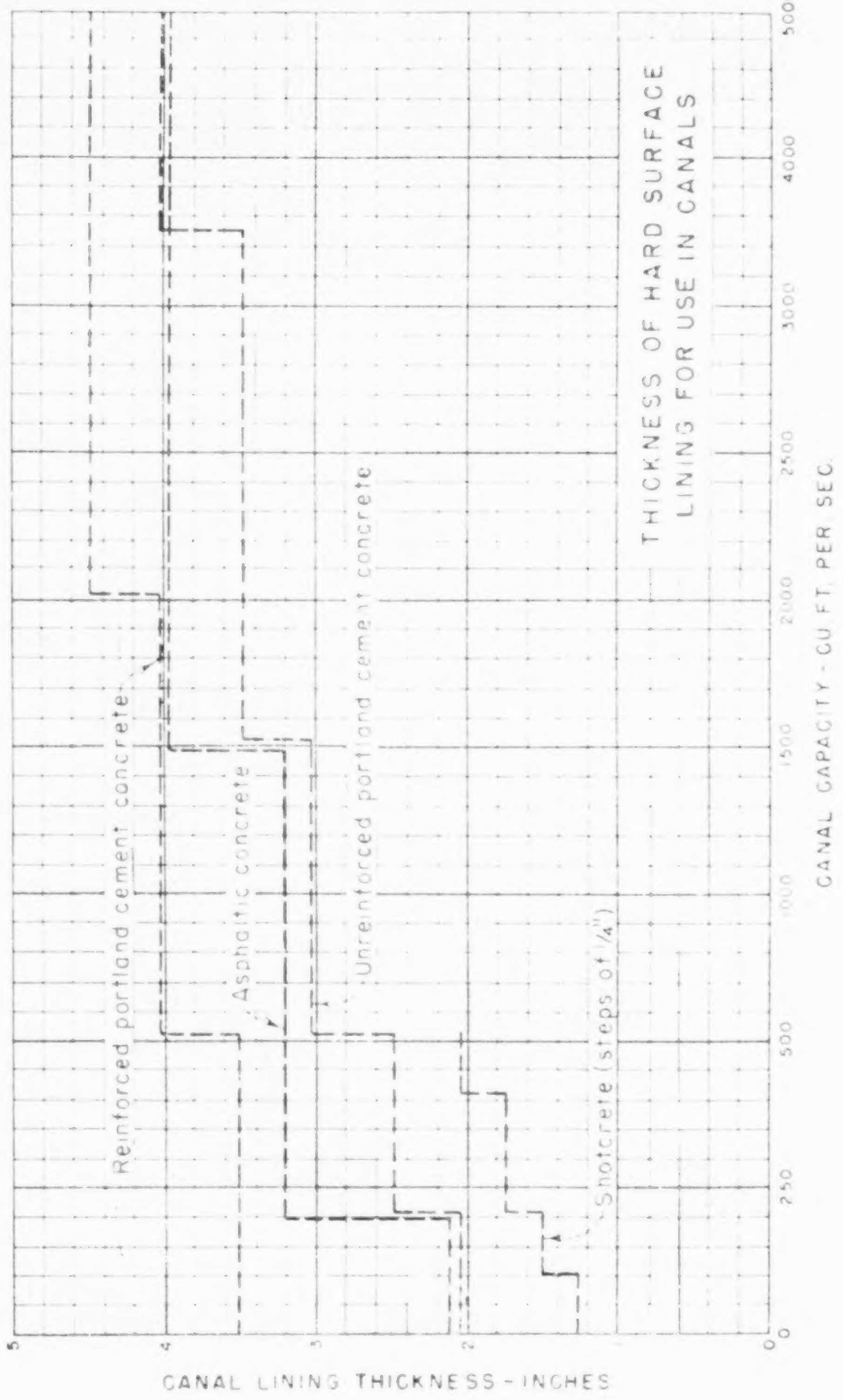
ALTERNATE A

ALTERNATE B

DETAILS OF JOINTS AT STRUCTURES

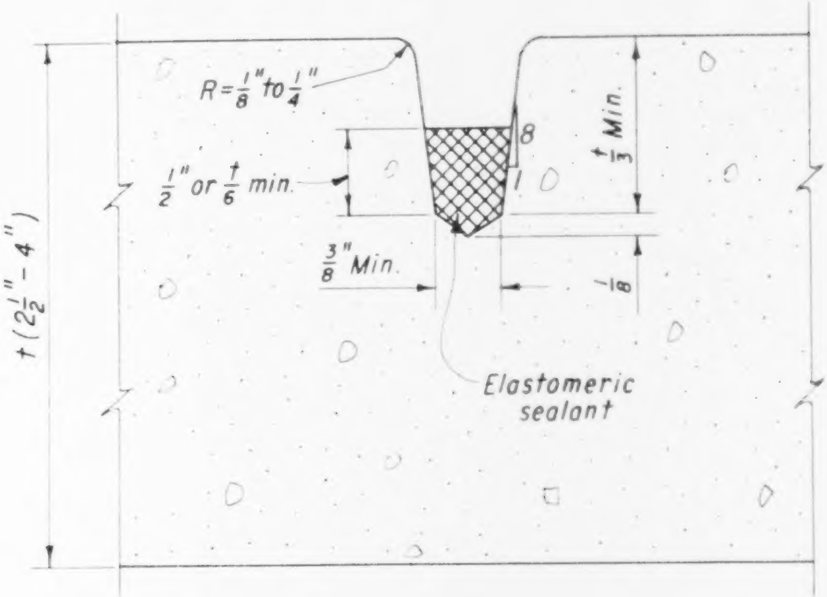
CONCRETE LINING SECTION - SHEET 2

Figure 9

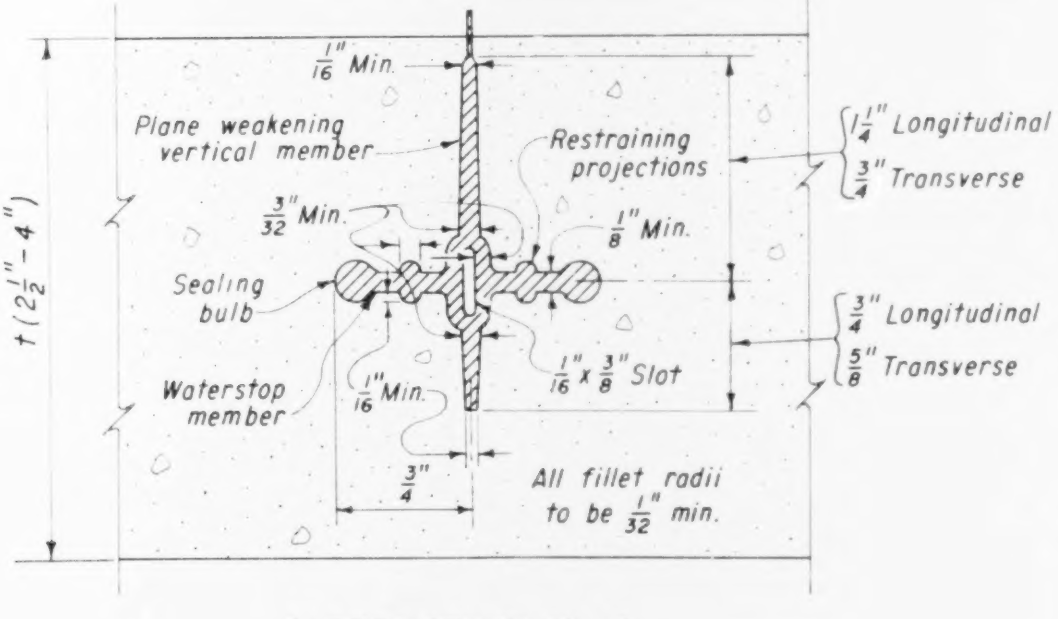


THICKNESS OF HARD - SURFACED LINING FOR USE IN CANALS

Figure 10

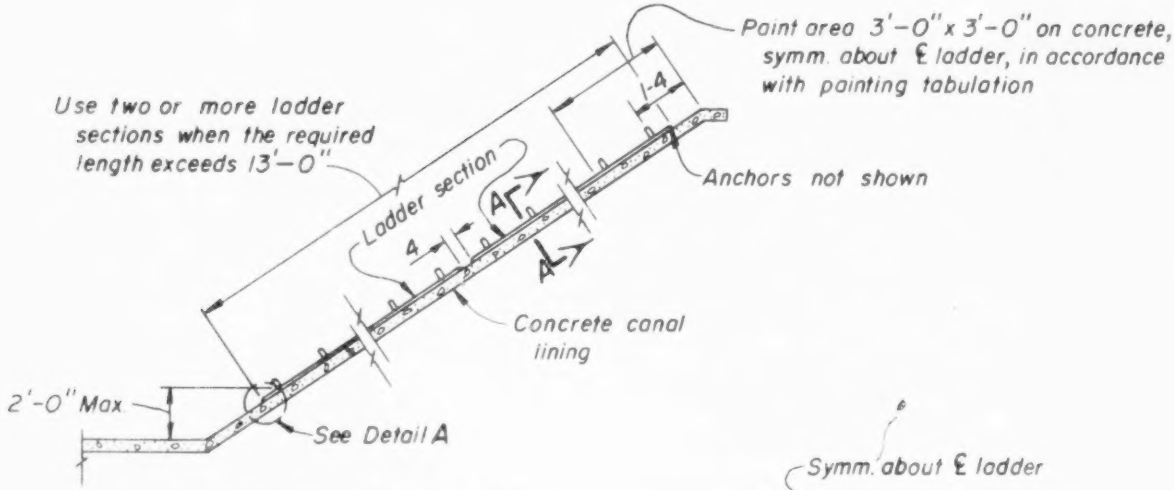

R = 1/8" to 1/4"
1/2" or t/6 min.
8
1
t/3 Min.
3/8" Min.
1/8
Elastomeric sealant
t (2 1/2" - 4")

ALTERNATIVE No. 1
(ELASTOMERIC SEALANT)

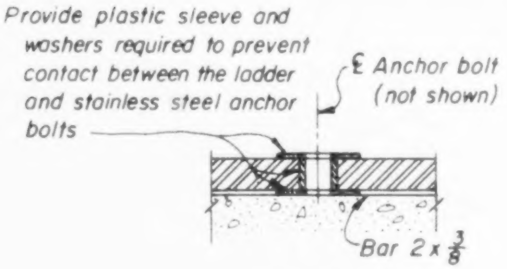

1/16" Min.
Plane weakening vertical member
Restraining projections
1 1/4" Longitudinal
3/4" Transverse
3/32" Min.
1/8" Min.
Sealing bulb
3/4" Longitudinal
5/8" Transverse
1/16" x 3/8" Slot
Waterstop member
1/16" Min.
3/4"
All fillet radii to be 1/32" min.
t (2 1/2" - 4")

ALTERNATIVE No. 2
(POLYVINYL CHLORIDE STRIP)

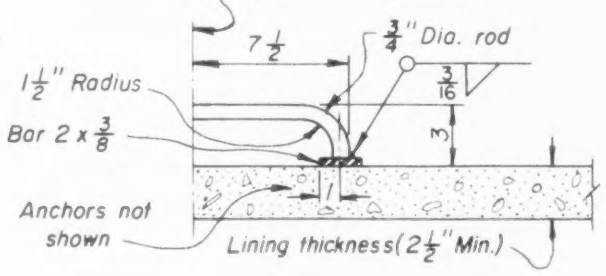
CONTRACTION JOINTS FOR UNREINFORCED LINING


Paint area 3'-0" x 3'-0" on concrete, symm. about ℄ ladder, in accordance with painting tabulation
Use two or more ladder sections when the required length exceeds 13'-0"
1-4
Ladder section
Anchors not shown
4
A
A
Concrete canal lining
2'-0" Max.
See Detail A

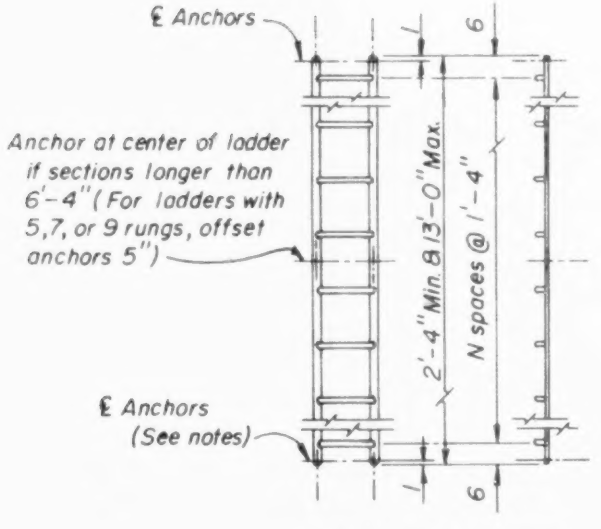
INSTALLATION DETAIL


Provide plastic sleeve and washers required to prevent contact between the ladder and stainless steel anchor bolts
℄ Anchor bolt (not shown)
Bar 2 x $\frac{3}{8}$

DETAIL A


Symm. about ℄ ladder
7 $\frac{1}{2}$
$\frac{3}{4}$" Dia. rod
1 $\frac{1}{2}$" Radius
$\frac{3}{16}$
Bar 2 x $\frac{3}{8}$
3
1
Anchors not shown
Lining thickness(2 $\frac{1}{2}$" Min.)

SECTION A-A
LADDER RUNG DETAIL


℄ Anchors
6
Anchor at center of ladder if sections longer than 6'-4" (For ladders with 5,7, or 9 rungs, offset anchors 5")
2'-4" Min. & 13'-0" Max.
N spaces @ 1'-4"
1
℄ Anchors (See notes)
1
6

LADDER SECTION

NOTES

Ladders to be installed on sides of canal when the vertical lining height is 2'-6" or greater.

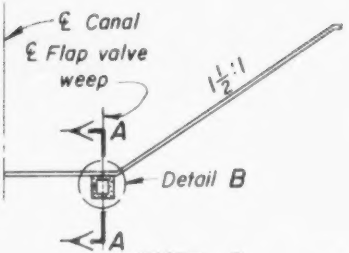
Ladders to be located opposite each other at 750-foot intervals along the canal and upstream of structures as directed.

Ladders to be fabricated from 6061-T6 aluminum unless otherwise specified.

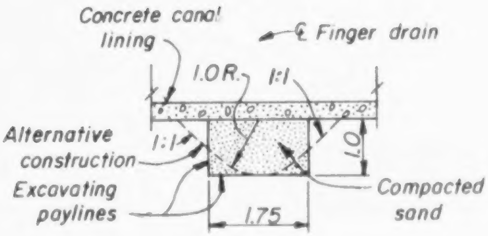
Ladders shall be anchored to the canal lining with $\frac{3}{4}$" Dia. by 2 $\frac{1}{2}$" long stainless steel expansion type or impact type anchors.

SAFETY LADDERS FOR CONCRETE LINED CANALS

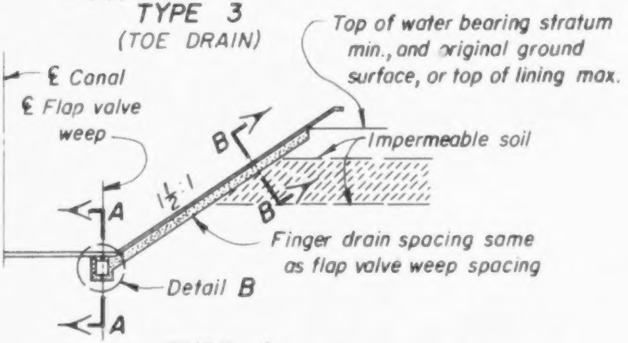
Figure 12


℄ Canal
℄ Flap valve weep
1 1/2:1
A
Detail B
A

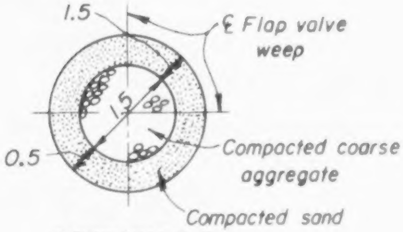
TYPE 3
(TOE DRAIN)


Concrete canal lining
℄ Finger drain
1.0 R.
1:1
Alternative construction
1:1
Excavating paylines
1.0
1.75
Compacted sand

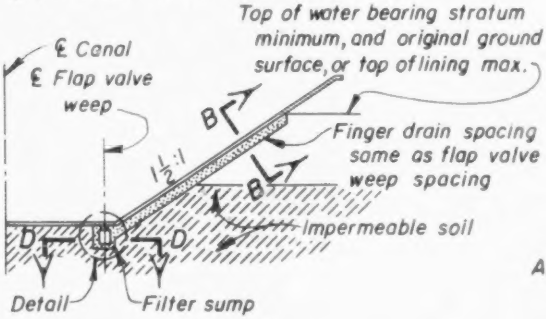
SECTION B-B


℄ Canal
℄ Flap valve weep
Top of water bearing stratum min., and original ground surface, or top of lining max.
Impermeable soil
B
B
1 1/2:1
A
Finger drain spacing same as flap valve weep spacing
Detail B
A

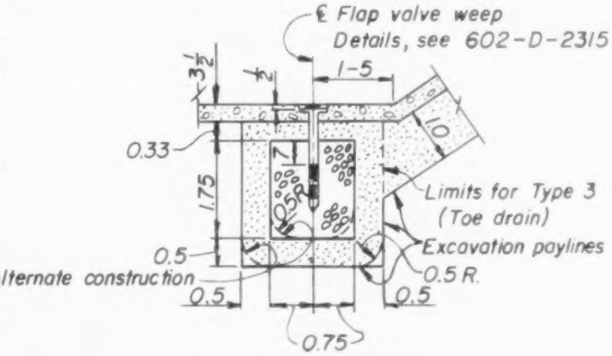
TYPE 4
(TOE DRAIN WITH FINGER DRAINS)


1.5
℄ Flap valve weep
1.5
0.5
Compacted coarse aggregate
Compacted sand

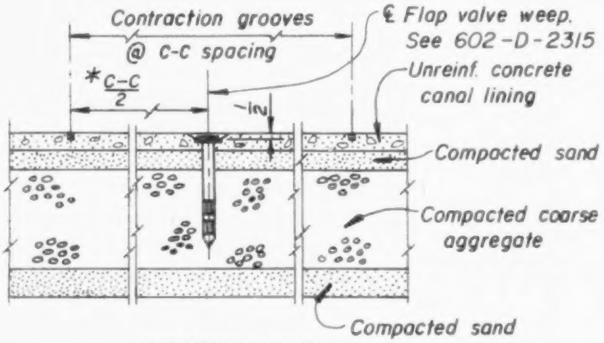
SECTION D-D


℄ Canal
℄ Flap valve weep
Top of water bearing stratum minimum, and original ground surface, or top of lining max.
B
Finger drain spacing same as flap valve weep spacing
1 1/2:1
B
Impermeable soil
D
D
Detail
Filter sump

TYPE 5
(FINGER DRAIN)


℄ Flap valve weep
Details, see 602-D-2315
3 1/2
1/2
1-5
1.0
0.33
1.75
0.5R
Limits for Type 3 (Toe drain)
0.5
Excavation paylines
Alternate construction
0.5 R.
0.5
0.5
0.75

DETAIL B


Contraction grooves @ C-C spacing
℄ Flap valve weep. See 602-D-2315
*C-C/2
1/2
Unreinf. concrete canal lining
Compacted sand
Compacted coarse aggregate
Compacted sand

SECTION A-A

*Adjust if two weeps per panel are required.

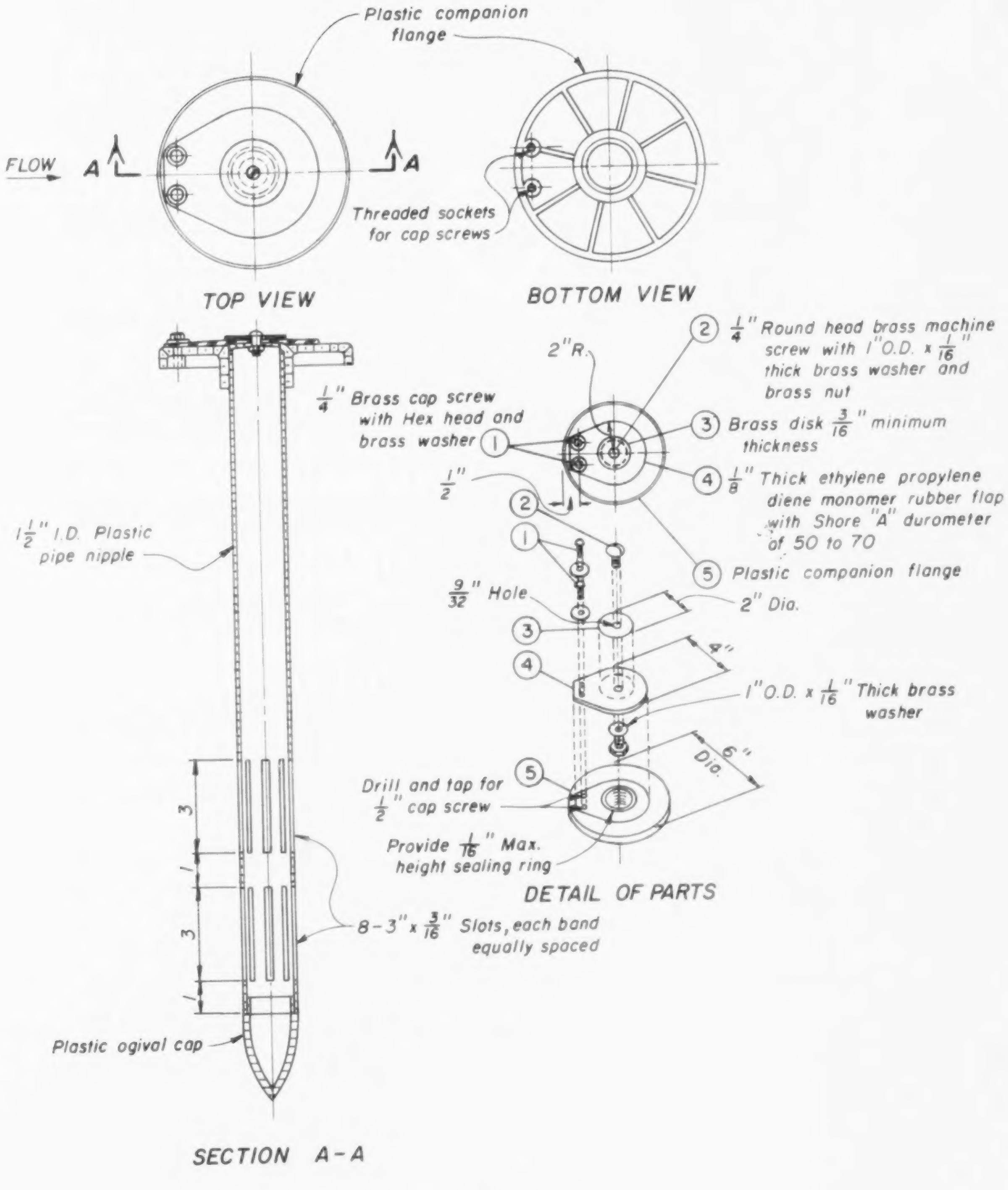
**Gradation should be checked based on gradation of base material.

****GRADATION OF FILTER MATERIAL**

FILTER	PERCENT (BY WEIGHT) RETAINED ON STANDARD SIEVE						
MATERIAL	#30	#16	#8	#4	$\frac{3}{8}$	$\frac{3}{4}$	$1\frac{1}{2}$
Sand	95-100	50-80	20-50	5-15			
Coarse Aggregate				95-100	50-80	20-50	0

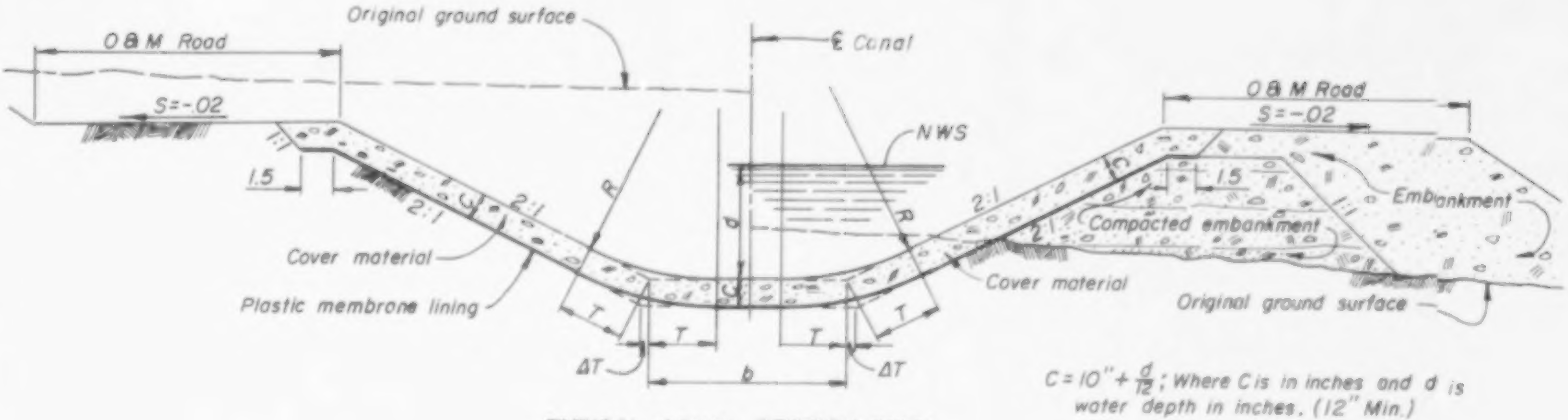
UNDERDRAINS FOR CONCRETE LINING

Figure 13


Plastic companion flange
FLOW
A
A
Threaded sockets for cap screws
TOP VIEW
BOTTOM VIEW
② 1/4" Round head brass machine screw with 1" O.D. x 1/16" thick brass washer and brass nut
2" R.
1/4" Brass cap screw with Hex head and brass washer ①
③ Brass disk 3/16" minimum thickness
④ 1/8" Thick ethylene propylene diene monomer rubber flap with Shore "A" durometer of 50 to 70
1/2"
②
①
1 1/2" I.D. Plastic pipe nipple
⑤ Plastic companion flange
9/32" Hole
2" Dia.
③
④
4"
1" O.D. x 1/16" Thick brass washer
6" Dia.
⑤
Drill and tap for 1/2" cap screw
Provide 1/16" Max. height sealing ring
DETAIL OF PARTS
3
1
3
1
8-3" x 3/16" Slots, each band equally spaced
Plastic ogival cap
SECTION A-A

FLAP VALVE WEEP

Figure 14


O & M Road
S=-.02
Original ground surface
℄ Canal
NWS
1:1
1.5
2:1
C
R
d
T
ΔT
b
Cover material
Plastic membrane lining
O & M Road
S=-.02
1.5
1:1
Embankment
Compacted embankment
Original ground surface

$C = 10'' + \frac{d}{12}$; Where C is in inches and d is water depth in inches. (12" Min.)

TYPICAL CANAL SECTION WITH PLASTIC MEMBRANE LINING

NOTES

The gradation, type and thickness of the cover material is dependant on tractive forces and velocities in the section, and the type of material available in the area.

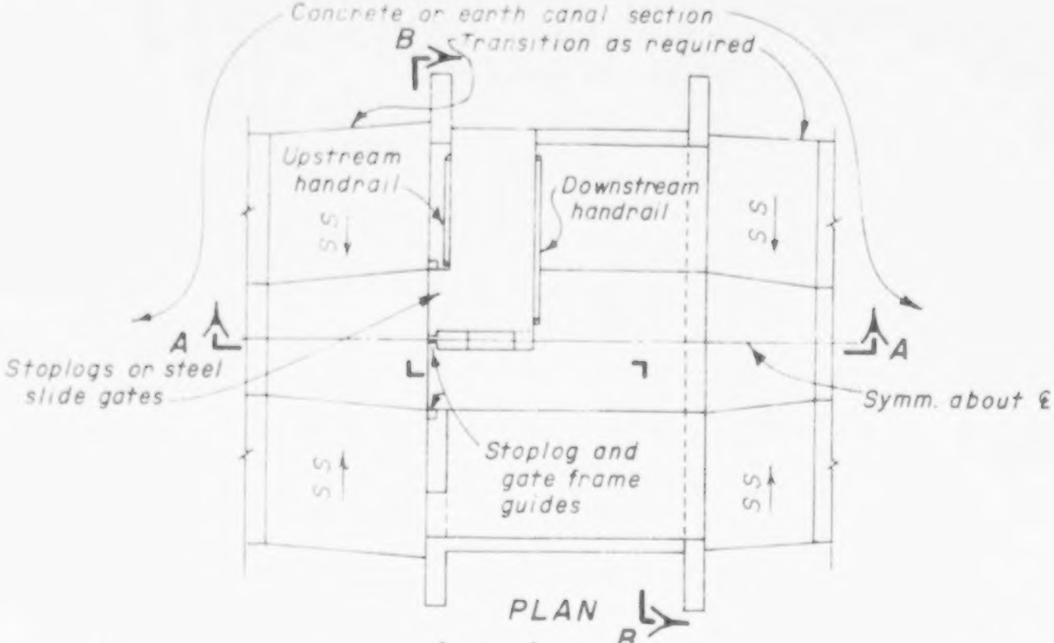
ΔWP is the difference between 2T and the arc length. To obtain the wetted perimeter for a section, subtract 2ΔWP from the wetted perimeter for a trapezoidal section of width b.

A is the area of the fillet. To obtain the area for a section, subtract 2A from the area of a trapozoidal section of width b.

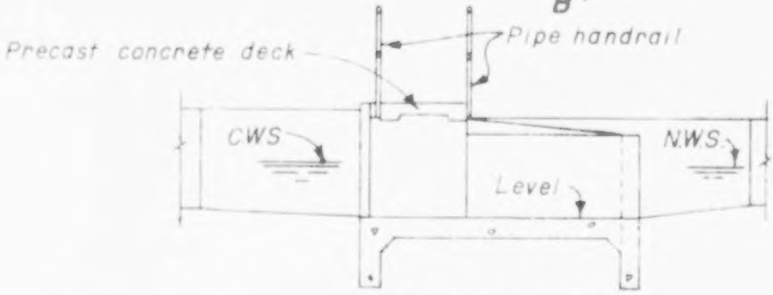
b	T	S:S=2:1, R=4.23608T ΔWP=.00850R, A=.00425 R^2			S:S=$2\frac{1}{2}$:1, R=5.19259T ΔWP=.00466R, A=.00233R^2			S:S=3:1, R=6.16226T ΔWP=.00280R, A=.00140 R^2		
		R	2ΔWP	2A	R	2ΔWP	2A	R	2ΔWP	2A
2	1.0	4.24	.07	.15	5.19	.05	.13	6.16	.03	.11
3	1.0	4.24	.07	.15	5.19	.05	.13	6.16	.03	.11
4	1.0	4.24	.07	.15	5.19	.05	.13	6.16	.03	.11
5	1.5	6.35	.11	.34	7.79	.07	.28	9.24	.05	.24
6	2.0	8.47	.14	.61	10.39	.10	.50	12.32	.07	.42
8	2.8	11.86	.20	1.20	14.54	.14	.99	17.25	.10	.83
10	3.5	14.83	.25	1.87	18.17	.17	1.54	21.57	.12	1.30
12	4.3	18.22	.31	2.82	22.33	.21	2.32	26.50	.15	1.97
14	5.0	21.18	.36	3.81	25.96	.24	3.14	30.81	.17	2.66
16	5.5	23.30	.40	4.61	28.56	.27	3.80	33.89	.19	3.22
18	6.0	25.42	.43	5.49	31.16	.29	4.52	36.97	.21	3.83
20	6.5	27.53	.47	6.44	33.75	.31	5.31	40.05	.22	4.49
22	7.0	29.65	.50	7.47	36.35	.34	6.16	43.14	.24	5.21
24	7.5	31.77	.54	8.58	38.94	.36	7.07	46.22	.26	5.98
26	7.9	33.47	.57	9.52	41.02	.38	7.84	48.68	.27	6.64
28	8.3	35.16	.60	10.51	43.10	.40	8.66	51.15	.29	7.33
30	8.7	36.85	.63	11.54	45.18	.42	9.51	53.61	.30	8.05
35	9.5	40.24	.68	13.76	49.33	.46	11.34	58.54	.33	9.60
40	10.5	44.48	.76	16.82	54.52	.51	13.85	64.70	.36	11.72
45	11.2	47.44	.81	19.13	58.16	.54	15.76	69.02	.39	13.34
50	12.0	50.83	.86	21.96	62.31	.58	18.09	73.95	.41	15.31

CANAL SECTION WITH PLASTIC MEMBRANE LINING

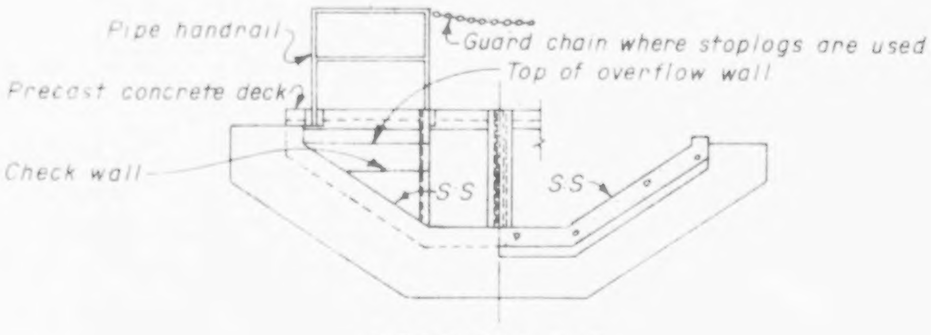
Figure 15


Concrete or earth canal section
B
Transition as required
Upstream handrail
Downstream handrail
S.S.
S.S.
S.S.
S.S.
A
A
Stoplogs or steel slide gates
Symm. about ℄
Stoplog and gate frame guides
B

PLAN


Precast concrete deck
Pipe handrail
CWS
N.W.S.
Level

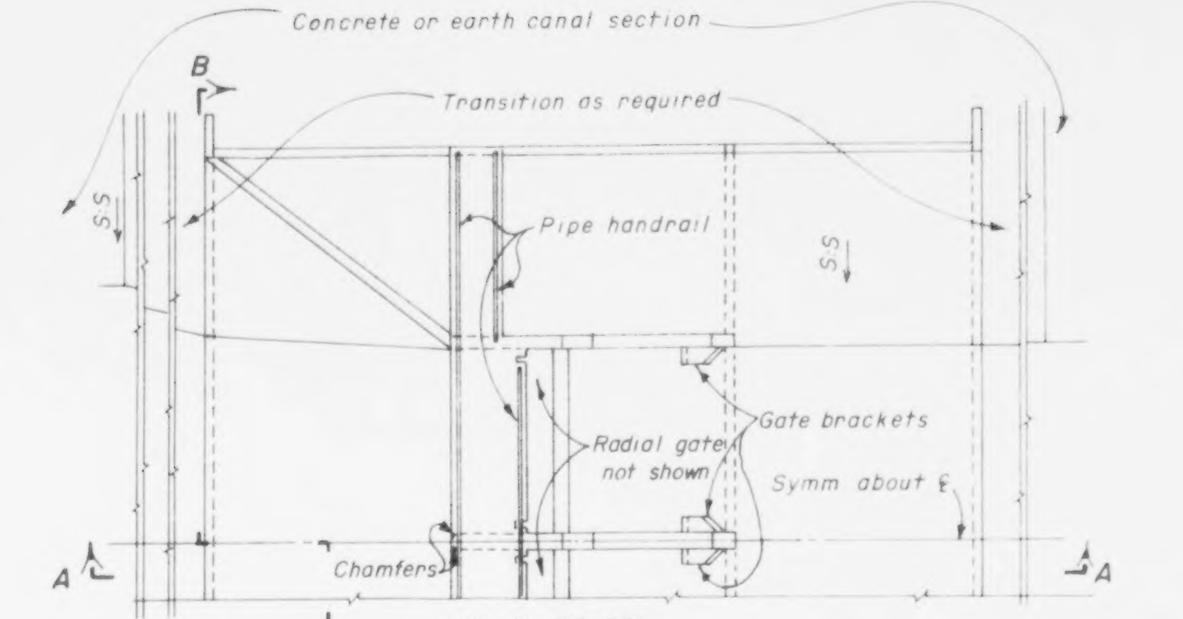
SECTION A-A


Pipe handrail
Guard chain where stoplogs are used
Top of overflow wall
Precast concrete deck
Check wall
S.S.
S.S.

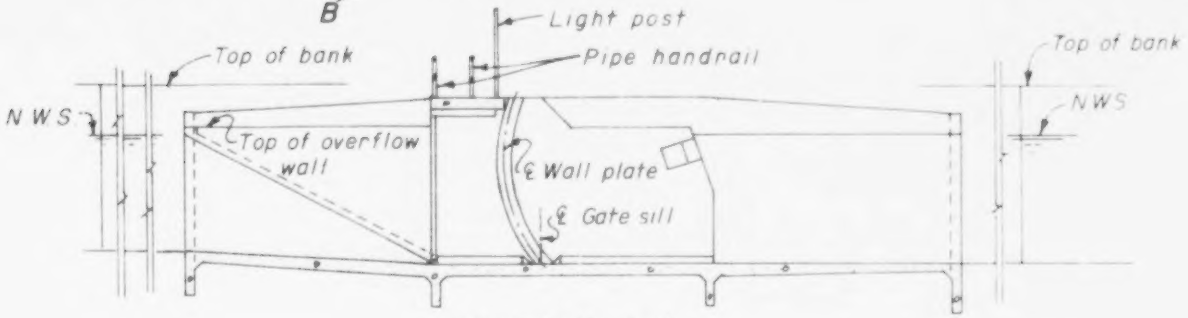
SECTION B-B

STOPLOG CHECK

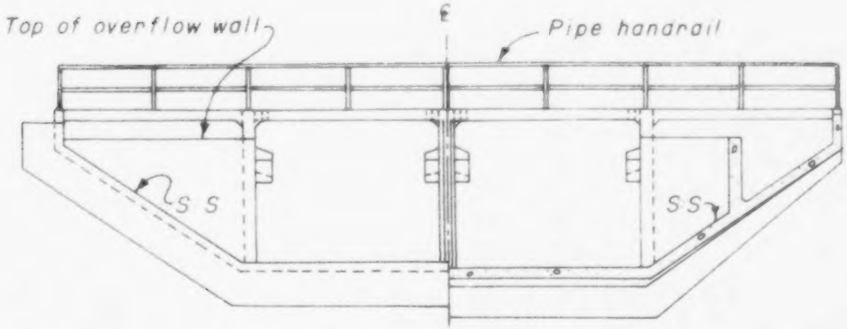
Figure 16


Concrete or earth canal section
B
Transition as required
S.S
Pipe handrail
S.S
Gate brackets
Radial gate not shown
Symm about ℄
A
Chamfers
A
B

HALF PLAN


Light post
Top of bank
Pipe handrail
Top of bank
N.W.S.
N.W.S.
Top of overflow wall
℄ Wall plate
℄ Gate sill

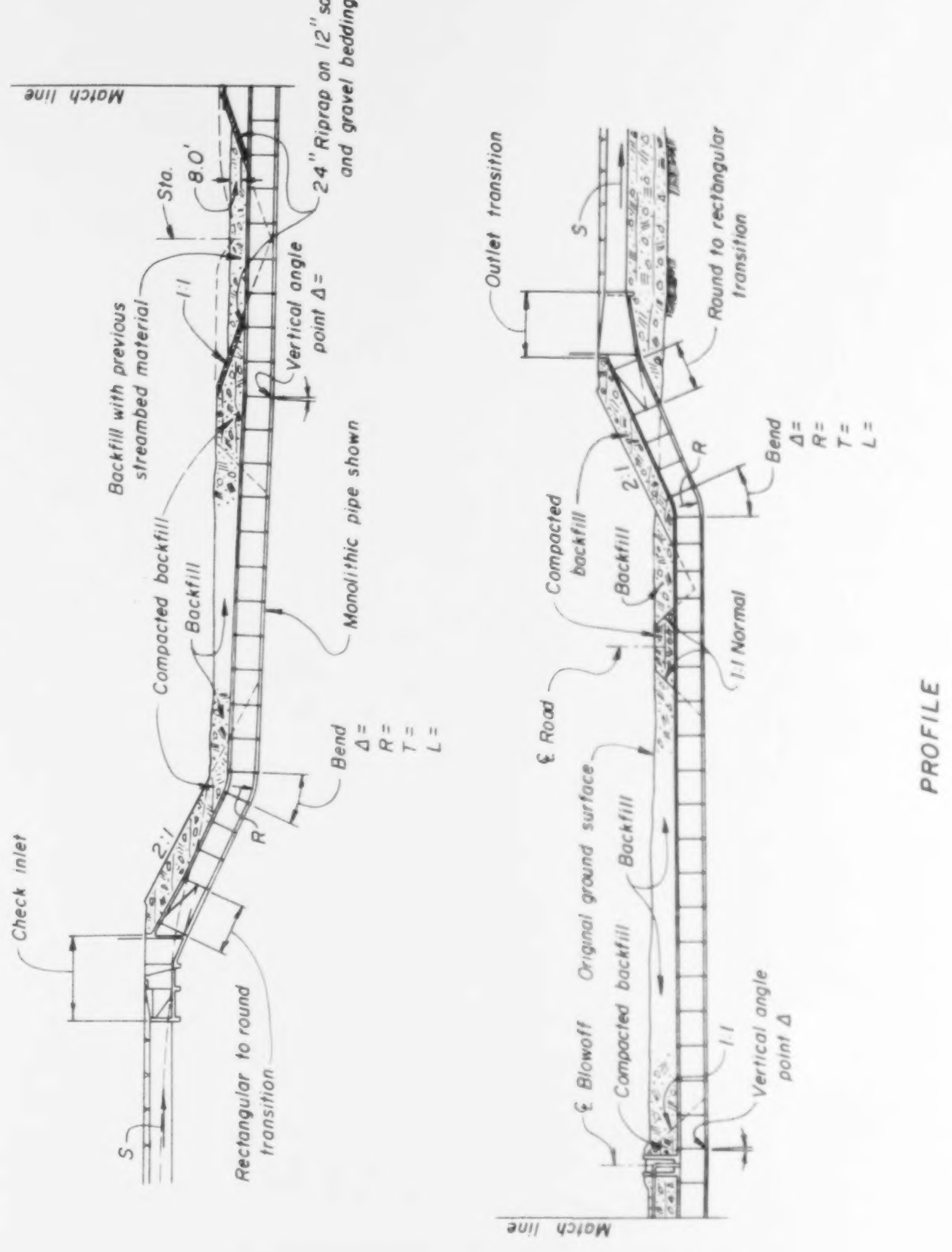
SECTION A-A


Top of overflow wall
℄
Pipe handrail
S.S
S.S

SECTION B-B

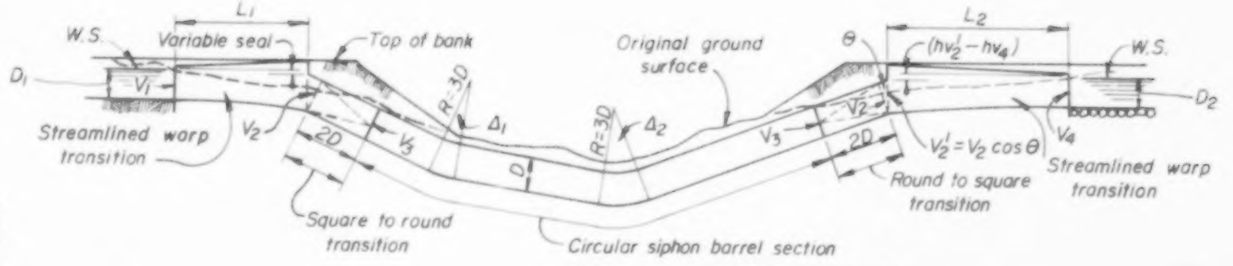
RADIAL GATE CHECK

ure 17


Match line
24" Riprap on 12" sand and gravel bedding
8.0'
Sta.
1:1
Backfill with previous streambed material
Vertical angle point Δ=
Monolithic pipe shown
Compacted backfill
Backfill
Bend
Δ=
R=
T=
L=
R
2:1
Check inlet
Rectangular to round transition
S
Match line
Outlet transition
S
Round to rectangular transition
Bend
Δ=
R=
T=
L=
R
2:1
Compacted backfill
Backfill
1:1 Normal
℄ Road
Original ground surface
Backfill
Compacted backfill
℄ Blowoff
1:1
Vertical angle point Δ
PROFILE

PIPE SIPHON

Figure 18


W.S.
D₁
V₁
L₁
Variable seal
Top of bank
Streamlined warp transition
V₂
2D
V₃
Square to round transition
R=3D
Δ₁
D
R=3D
Δ₂
Original ground surface
Circular siphon barrel section
V₃
2D
Round to square transition
θ
V₂
L₂
(hv'₂ − hv₄)
V'₂=V₂ cos θ
V₄
Streamlined warp transition
W.S.
D₂

PROFILE – TYPICAL SIPHON

EXAMPLE:

DIMENSIONS AND HYDRAULIC PROPERTIES

CANAL END OF INLET AND
OUTLET TRANSITIONS
Base Width = 25.0'
Side slopes = 1-1/2:1'
$d_1 = d_2 = 10.00'$
Q = 1000
A = 400.00
$V_1 = V_4 = 2.50$, $hv_1 = hv_4 = .097$
r = 6.55
n = .014
s = .00005-

SQUARE OPENING OF
CLOSED TRANSITIONS
Size - 11.0'x11.0'
Q = 1000
A = 121.00
$V_2 = 8.26$, $hv_2 = 1.061$
r = 2.75
n = .014
s = .00157

CIRCULAR SIPHON
BARREL
Dia. = 11.0'
Q = 1000
A = 95.03
$V_3 = 10.52$, $hv_3 = 1.721'$
r = 2.75
n = .013 (Steel form finish or equal)
s = .00220
n = .014
s = .00255

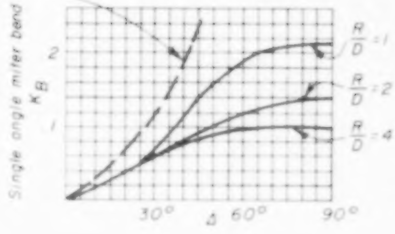
COMPUTATION OF HEAD LOSSES

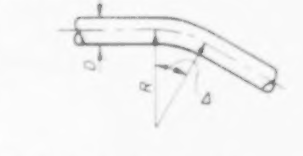
Inlet Open Transition (Friction)	$= 45'(L_1)(\frac{.00005+.00157}{2})$	=.036'
Inlet Open Transition (Convergence)	$= .1[1.061(hv_2)-.97(hv_1)]$	=.096'
Inlet Closed Transition (Convergence)	$= .1[1.721(hv_3)-1.061(hv_2)]$	=.066'
Closed Transitions (Friction)	$=2x22'(\frac{.00157+.00255}{2})$	=.092'
Circular Barrel (Friction)	=160'x.00220	=.352'
Barrel Bend $\Delta_1=15°$	$= .027x1.721(hv_3)$	=.046'
Barrel Bend $\Delta_2=30°$	$= .058x1.721(hv_3)$	=.100'
Outlet Closed Transition (Divergence)	$= .2[1.721(hv_3)-1.061(hv_2)]$	=.132'
Outlet Open Transition (Divergence)	$= .2[1.061(hv_2)-.097(hv_4)]$	=.193'
Outlet Open Transition (Friction)	$=60'(L_2)(\frac{.00005+.00157}{2})$	=.049'
Total Loss (Energy Gradient)		=1.161'
Add 10%± for excess capacity		= .116'
Total Head required		=1.277'

Type of open transition to closed conduit	Inlet	Outlet
Streamlined warp to rectangular opening....	0.1	0.2
Straight warp to rectangular opening.......	0.2	0.3
Straight warp with bottom corner fillets to pipe opening............................	0.1	0.4
Broken back to rectangular opening.........	0.3	0.5
Broken back to pipe opening................	0.4	0.7

Closed transition

Square or rectangular to round (maximum angle with centerline = 7-1/2°)............	0.1	0.2



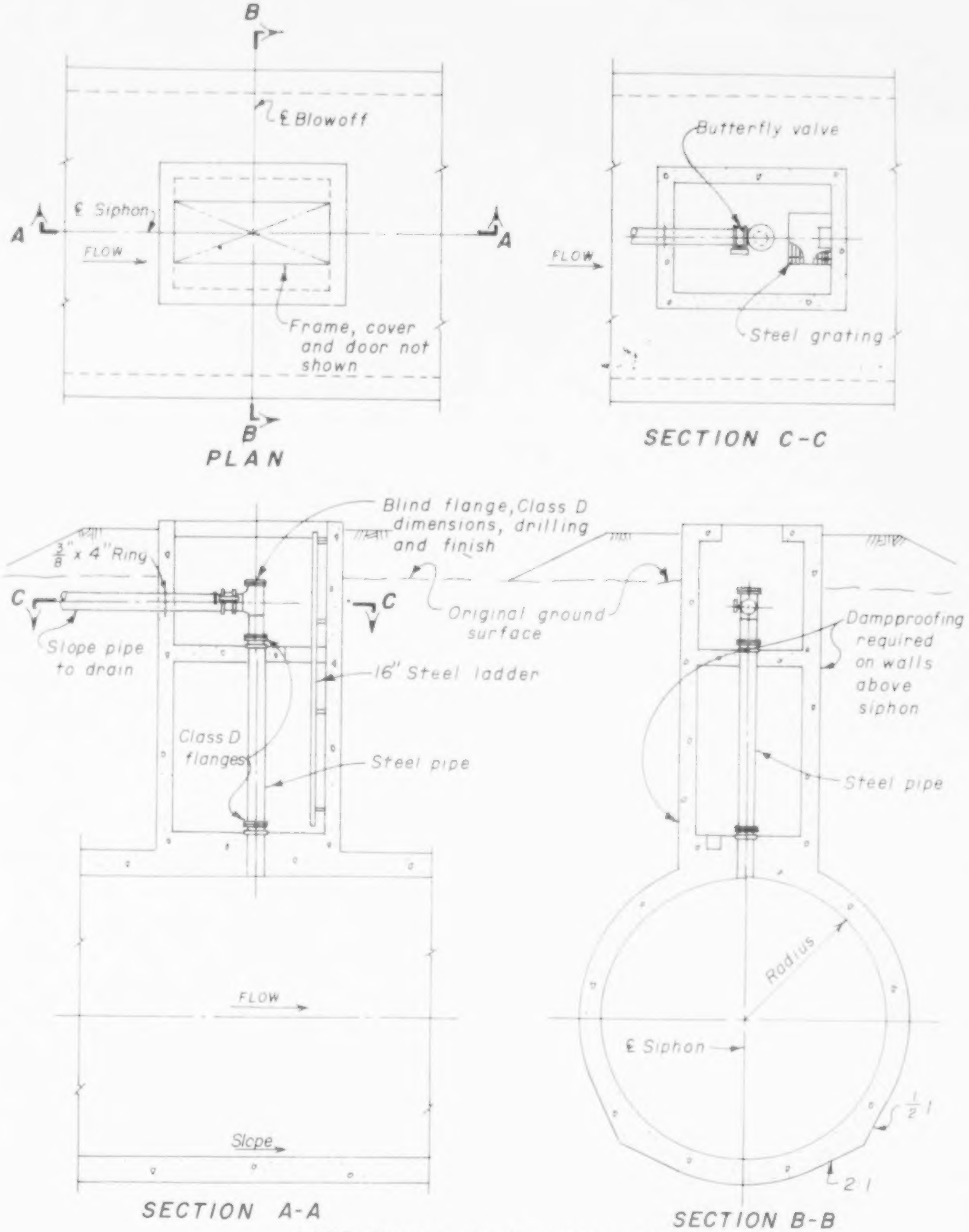


Δ= Bend deflection angle

K_B (For $h_L = K_B \frac{V^2}{2g}$)

HYDRAULIC COMPUTATIONS FOR PIPE SIPHON

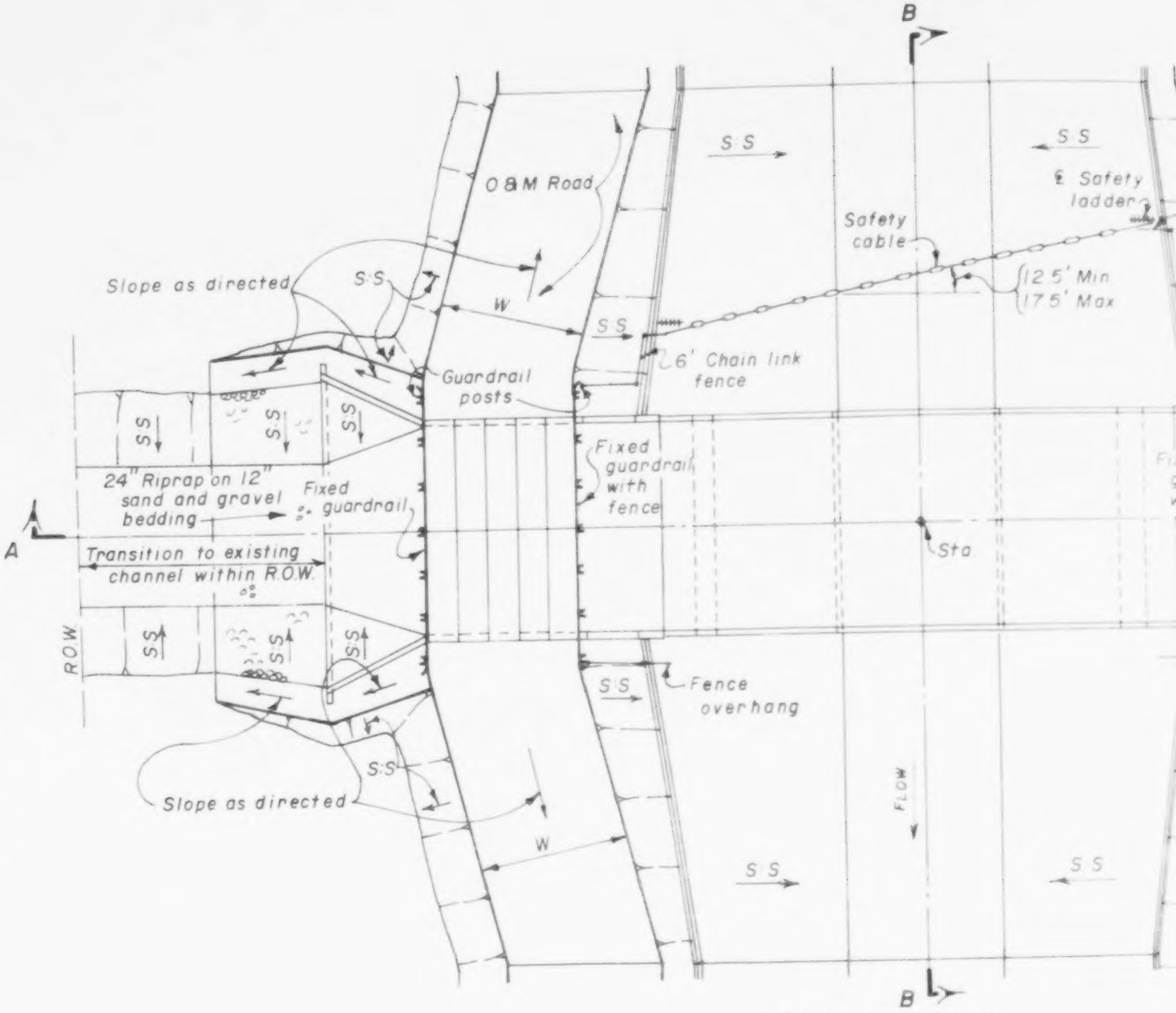
Figure 19


B
℄ Blowoff
℄ Siphon
A
A
FLOW
Frame, cover and door not shown
B
PLAN
Butterfly valve
FLOW
Steel grating
SECTION C-C
Blind flange, Class D dimensions, drilling and finish
$\frac{3}{8}$" x 4" Ring
C
C
Slope pipe to drain
Original ground surface
16" Steel ladder
Class D flanges
Steel pipe
FLOW
Slope
SECTION A-A
Dampproofing required on walls above siphon
Steel pipe
Radius
℄ Siphon
$\frac{1}{2}$:1
2:1
SECTION B-B

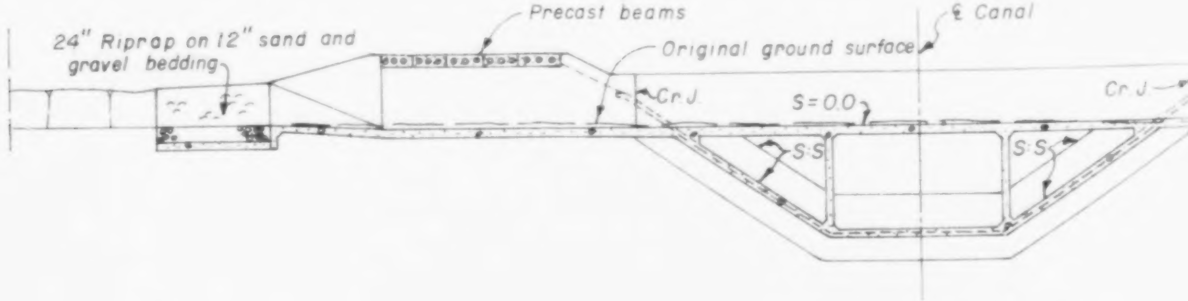
NOTE: Manhole may be placed in blowoff sump

BLOWOFF (MONOLITHIC PIPE)

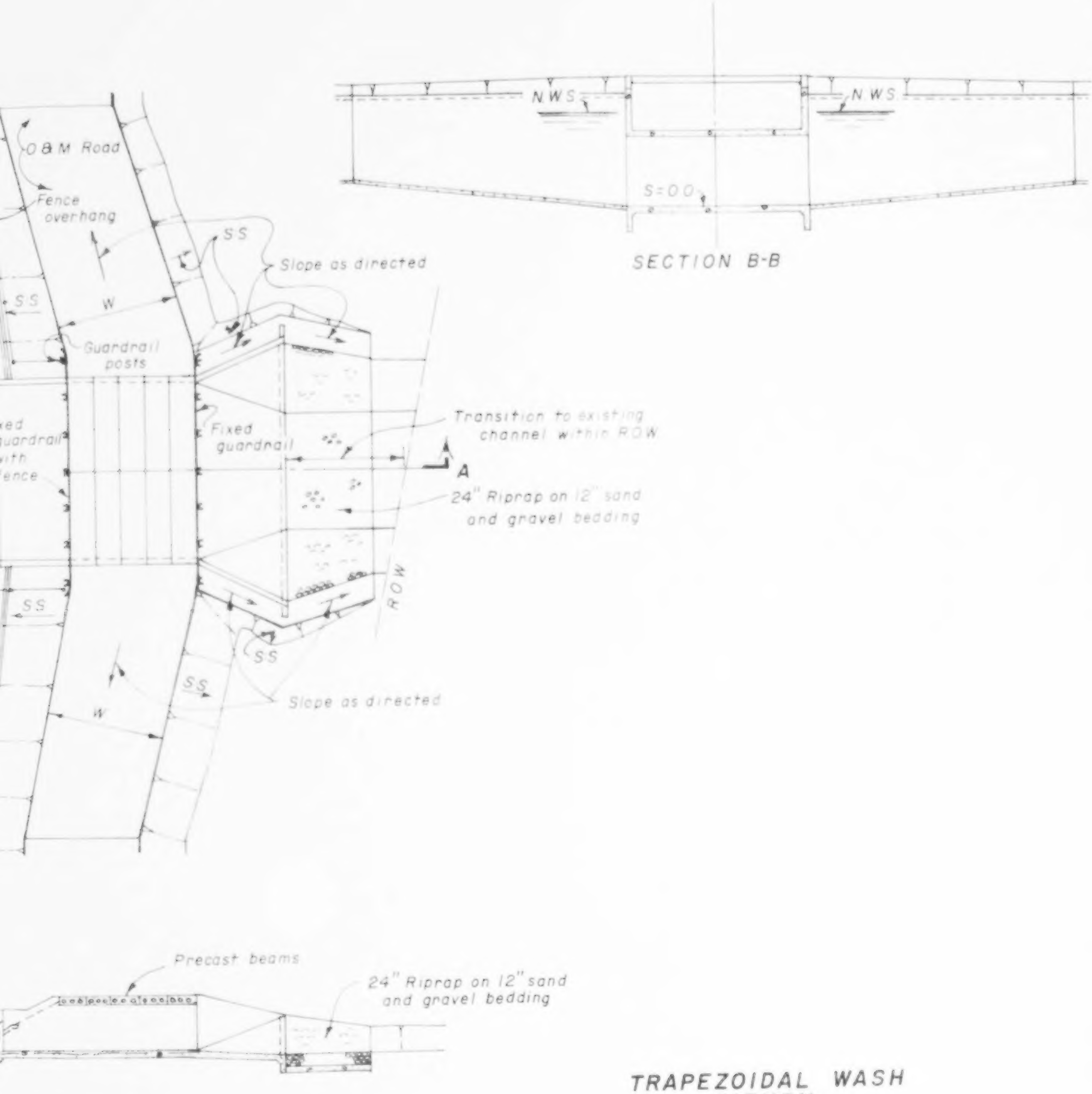
Figure 20


B
B
S·S
S·S
O & M Road
℄ Safety ladder
Safety cable
12.5' Min
17.5' Max
Slope as directed
S·S
W
S·S
6' Chain link fence
Guardrail posts
SS
S·S
S·S
Fixed guardrail with fence
24" Riprap on 12" sand and gravel bedding
Fixed guardrail
A
A
Transition to existing channel within R.O.W.
Sta
R.O.W.
SS
S·S
S·S
S·S
Fence overhang
S·S
Slope as directed
FLOW
W
S·S
S·S

GENERAL PLAN

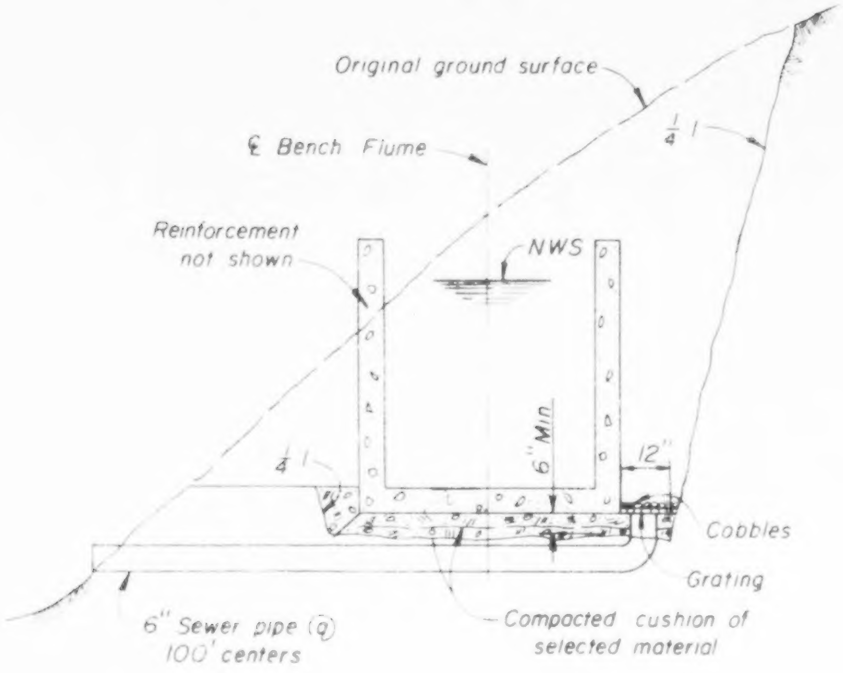

Precast beams
℄ Canal
24" Riprap on 12" sand and gravel bedding
Original ground surface
Cr. J.
S=0.0
Cr. J.
S·S
S·S

SECTION A-A

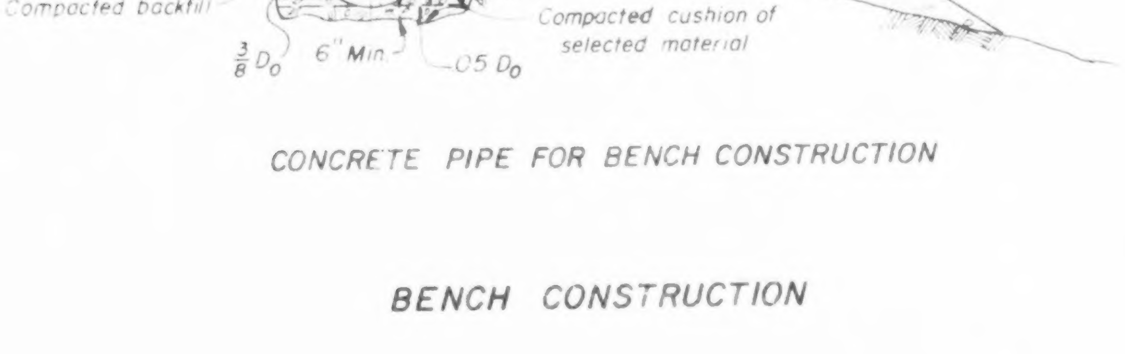

N.W.S.
N.W.S.
S=0.0
SECTION B-B
O&M Road
Fence overhang
S.S
Slope as directed
S.S
W
Guardrail posts
xed guardrail with fence
Fixed guardrail
Transition to existing channel within ROW
A
24" Riprap on 12" sand and gravel bedding
ROW
S.S
S.S
S.S
Slope as directed
W
Precast beams
24" Riprap on 12" sand and gravel bedding

TRAPEZOIDAL WASH SIPHON

Figure 21

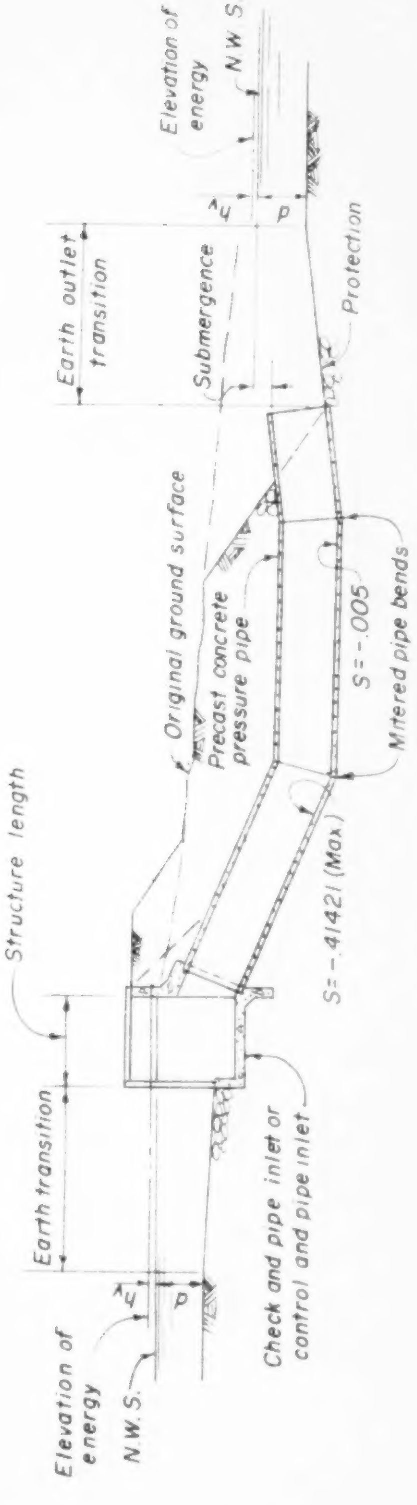

Original ground surface
℄ Bench Flume
$\frac{1}{4}$:1
Reinforcement not shown
NWS
6" Min
12"
$\frac{1}{4}$:1
Cobbles
Grating
6" Sewer pipe @ 100' centers
Compacted cushion of selected material

BENCH FLUME

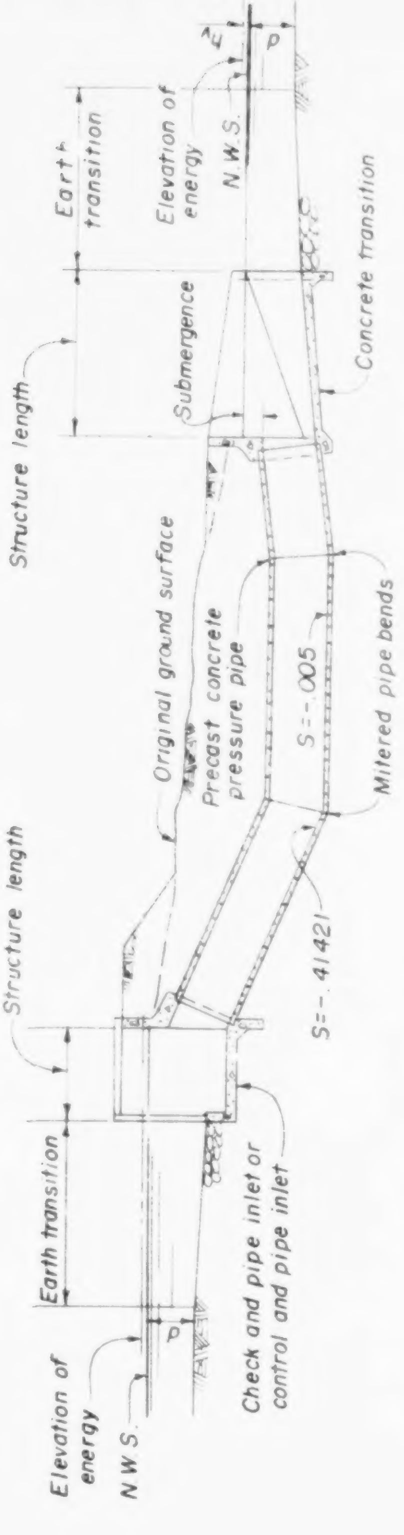

$\frac{D_o}{2}$
20:1
1$\frac{1}{2}$:1
Precast concrete pressure pipe
Original ground surface
Backfill
1:1
3.0' Min
Top of rock
1.5'
1.5'
12' Min. O&M Road
D_o
S=-.04
1$\frac{1}{2}$:1
$\frac{1}{4}$:1
Compacted backfill
Compacted cushion of selected material
$\frac{3}{8}D_o$
6" Min.
.05 D_o

CONCRETE PIPE FOR BENCH CONSTRUCTION

BENCH CONSTRUCTION


Elevation of energy
N.W.S.
h
d
Earth transition
Check and pipe inlet or control and pipe inlet
Structure length
S=-.41421 (Max)
Original ground surface
Precast concrete pressure pipe
S=-.005
Mitered pipe bends
Submergence
Protection
Earth outlet transition
h
d
Elevation of energy
N.W.S.

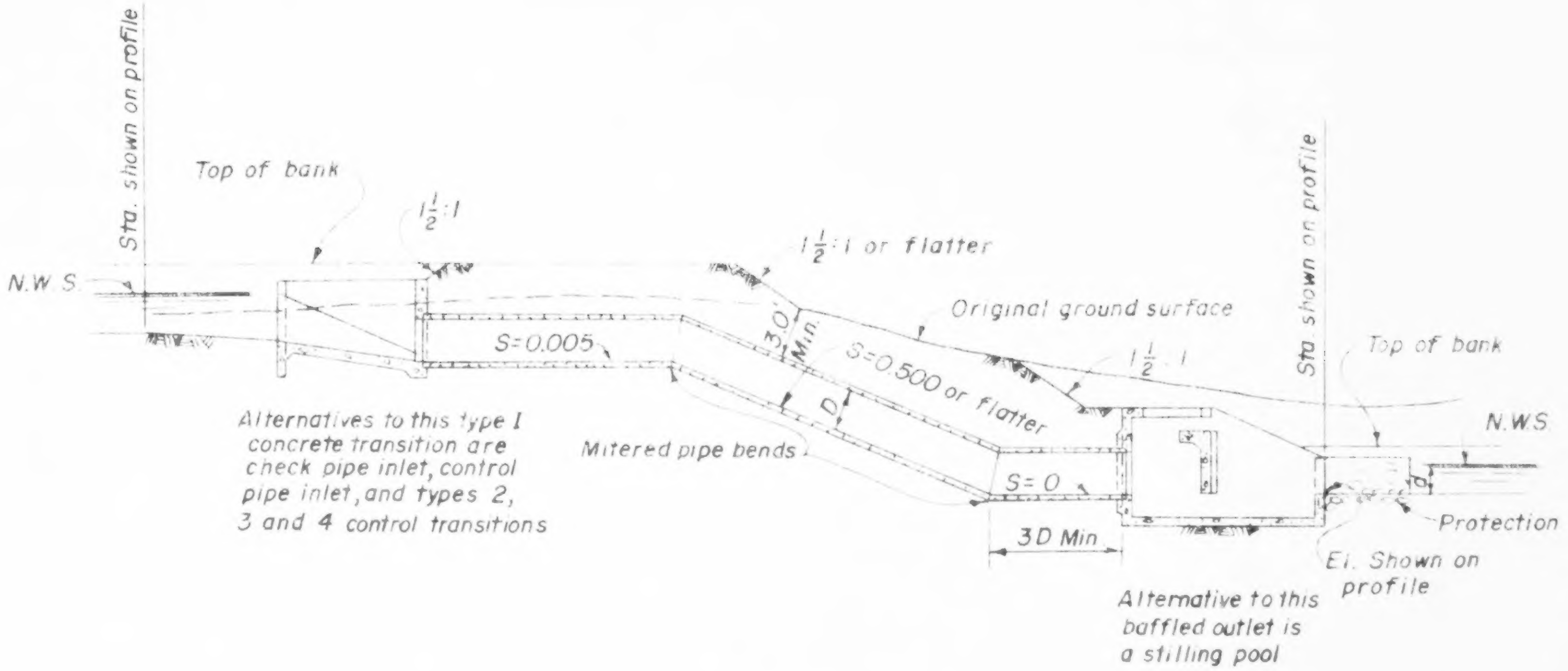
PROFILE WITHOUT OUTLET TRANSITION


Elevation of energy
N.W.S.
d
Earth transition
Check and pipe inlet or control and pipe inlet
Structure length
S=-.41421
Original ground surface
Precast concrete pressure pipe
S=-.005
Mitered pipe bends
Structure length
Submergence
Concrete transition
Earth transition
Elevation of energy
N.W.S.
h
d

PROFILE WITH OUTLET TRANSITION

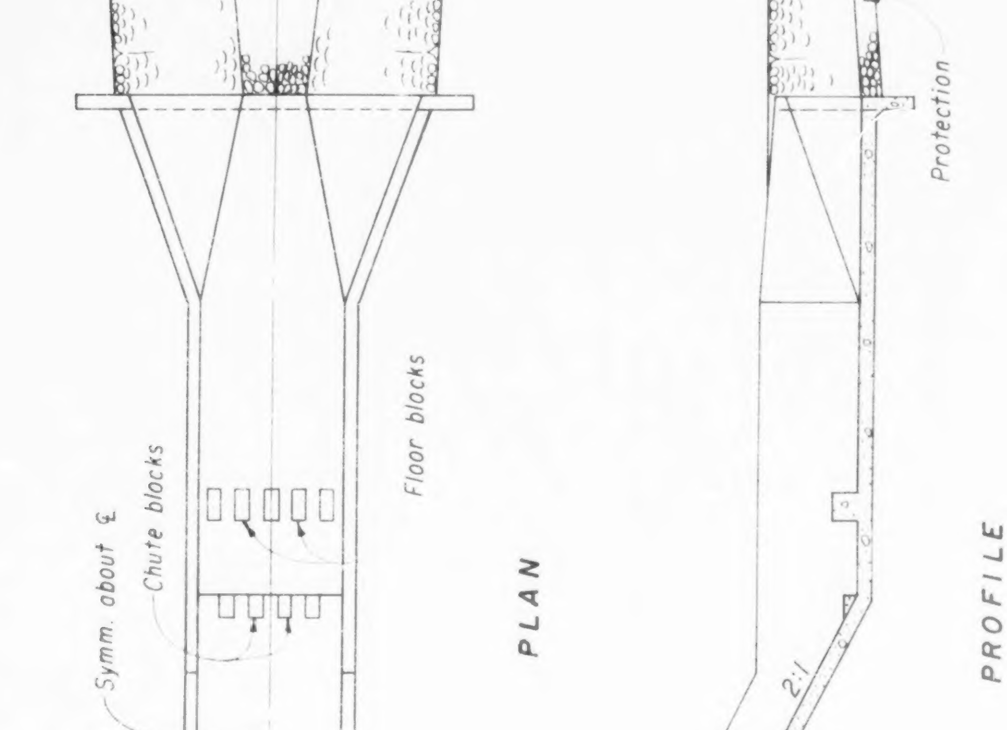
TYPE I PIPE DROPS

Figure 23

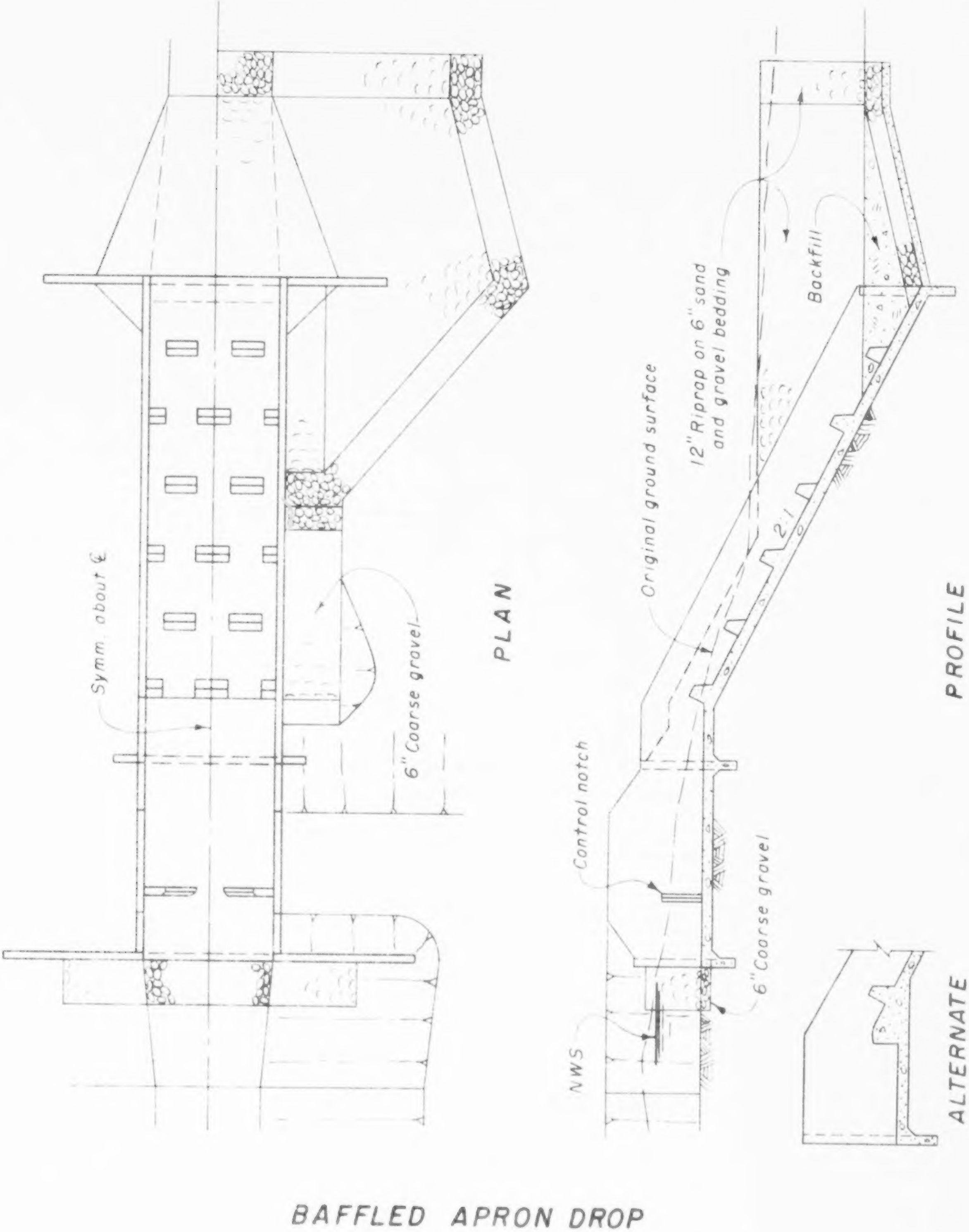

Sta. shown on profile
Top of bank
N.W.S.
$1\frac{1}{2}:1$
$1\frac{1}{2}:1$ or flatter
Original ground surface
3.0' Min.
S=0.005
S=0.500 or flatter
D
Alternatives to this type I concrete transition are check pipe inlet, control pipe inlet, and types 2, 3 and 4 control transitions
Mitered pipe bends
$1\frac{1}{2}:1$
Sta. shown on profile
Top of bank
N.W.S.
S=0
3D Min
Protection
El. Shown on profile
Alternative to this baffled outlet is a stilling pool

PROFILE

TYPE 2 PIPE DROP

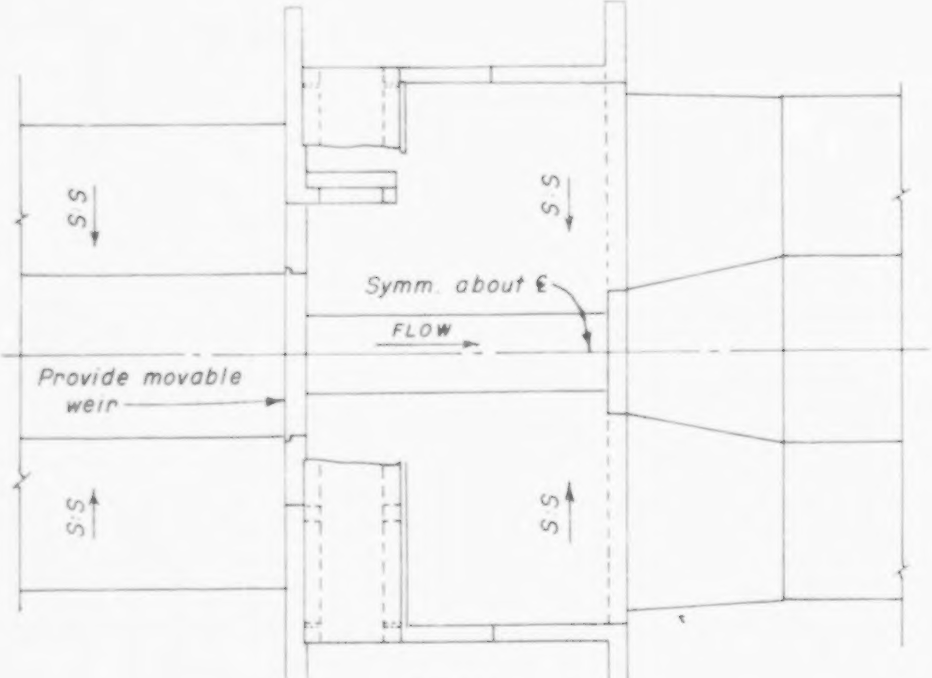

Gate frame guides
Symm. about ℄
Chute blocks
Floor blocks
PLAN
Slide gate not shown
Deck
2:1
Protection
PROFILE

RECTANGULAR INCLINED DROP

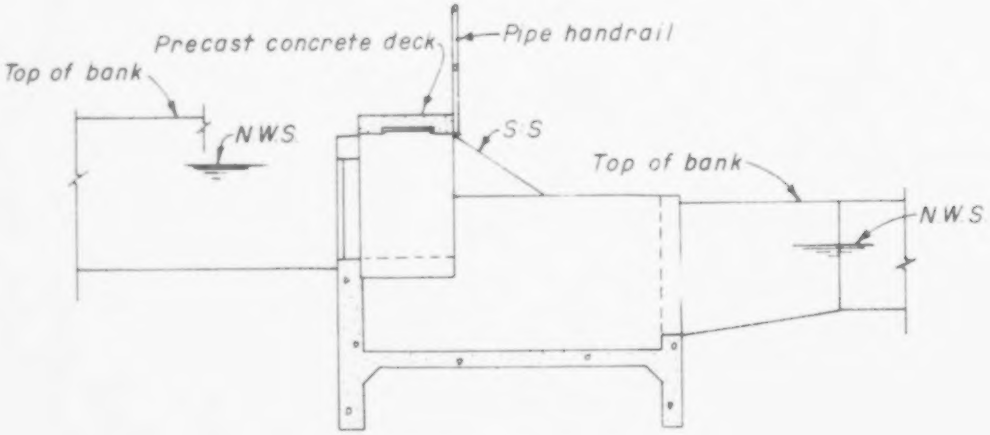

Symm. about ℄
6" Coarse gravel
PLAN
Original ground surface
12" Riprap on 6" sand and gravel bedding
Backfill
2:1
Control notch
6" Coarse gravel
NWS
PROFILE
ALTERNATE INLET

BAFFLED APRON DROP

Figure 26


S.S
S.S
Symm. about ℄
FLOW
Provide movable weir
S.S
S.S

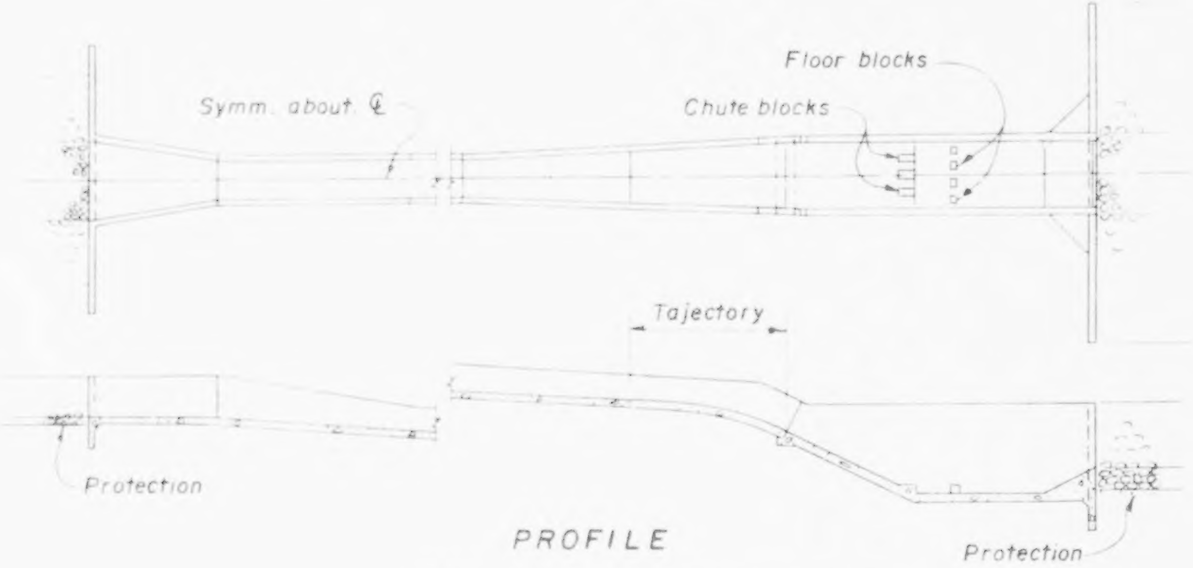
PLAN


Precast concrete deck
Pipe handrail
Top of bank
N.W.S.
S.S
Top of bank
N.W.S.

SECTION A-A

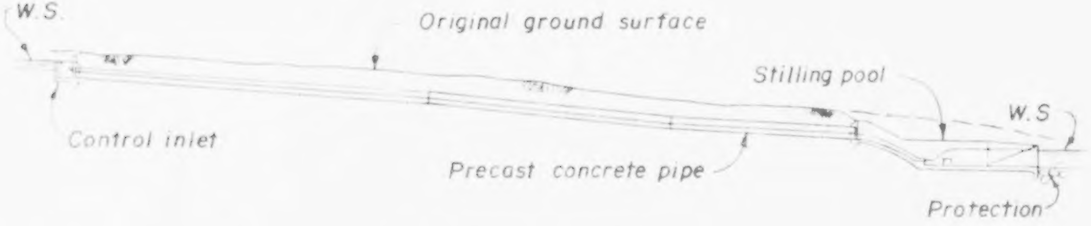
CHECK DROP

Figure 27

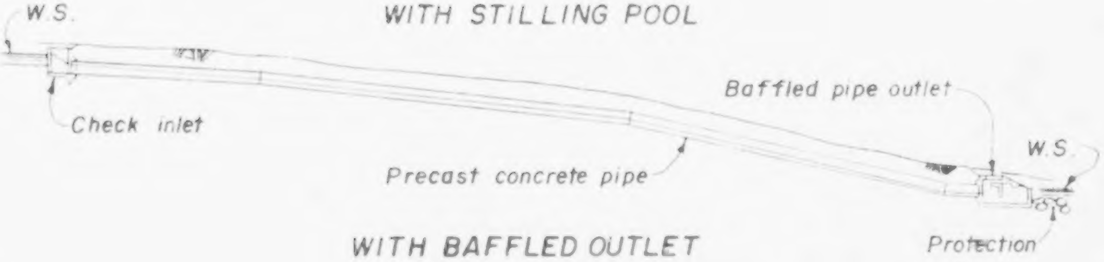

Floor blocks
Symm. about ℄
Chute blocks
Tajectory
Protection
Protection

PROFILE

RECTANGULAR CHUTE


W.S.
Original ground surface
Stilling pool
W.S
Control inlet
Precast concrete pipe
Protection

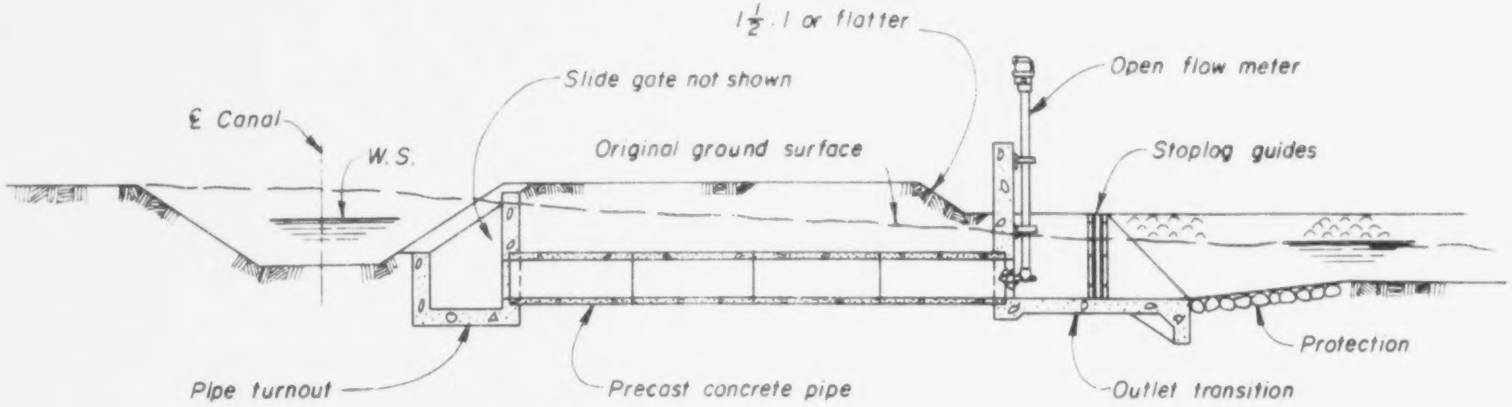
WITH STILLING POOL


W.S.
Baffled pipe outlet
Check inlet
W.S.
Precast concrete pipe
Protection

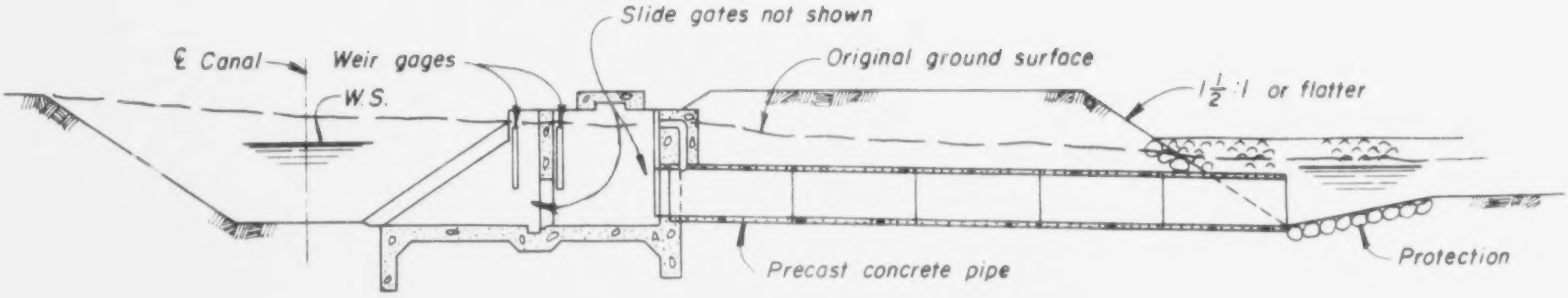
WITH BAFFLED OUTLET

PIPE CHUTES

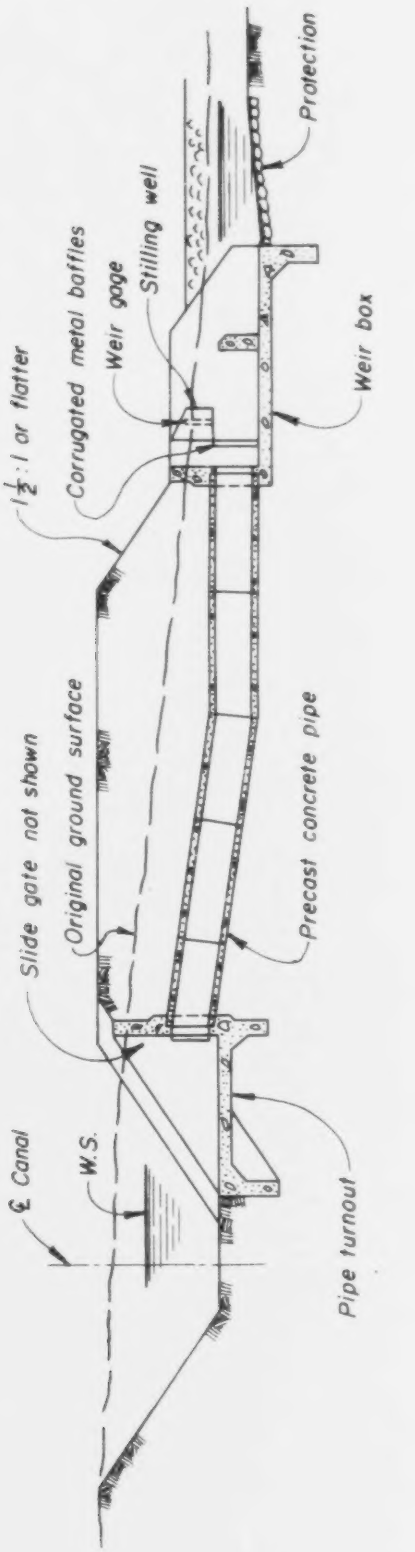
Figure 28


$1\frac{1}{2}:1$ or flatter
Open flow meter
Slide gate not shown
℄ Canal
W.S.
Original ground surface
Stoplog guides
Protection
Pipe turnout
Precast concrete pipe
Outlet transition

OPEN FLOW METER TURNOUT

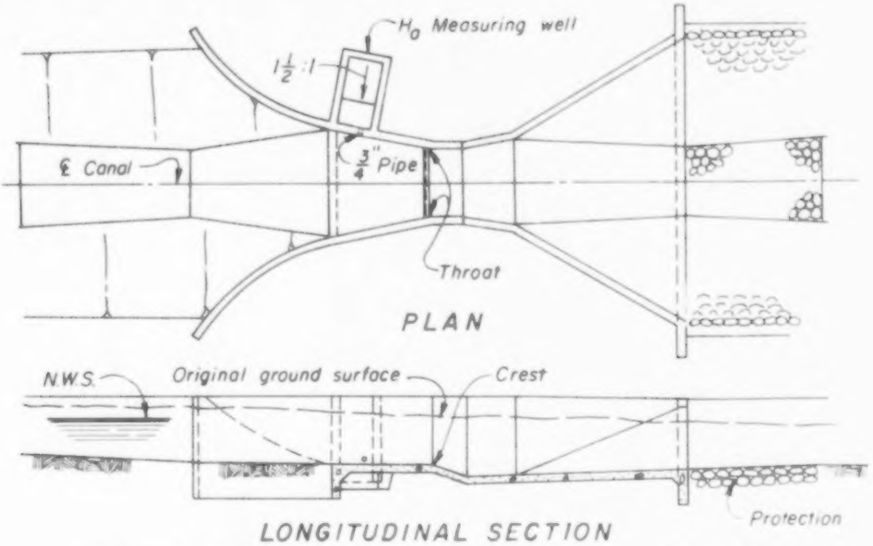

Slide gates not shown
℄ Canal
Weir gages
W.S.
Original ground surface
$1\frac{1}{2}:1$ or flatter
Protection
Precast concrete pipe

CONSTANT HEAD ORIFICE TURNOUT


℄ Canal
W.S.
Pipe turnout
Slide gate not shown
Original ground surface
Precast concrete pipe
$1\frac{1}{2}:1$ or flatter
Corrugated metal baffles
Weir gage
Stilling well
Weir box
Protection

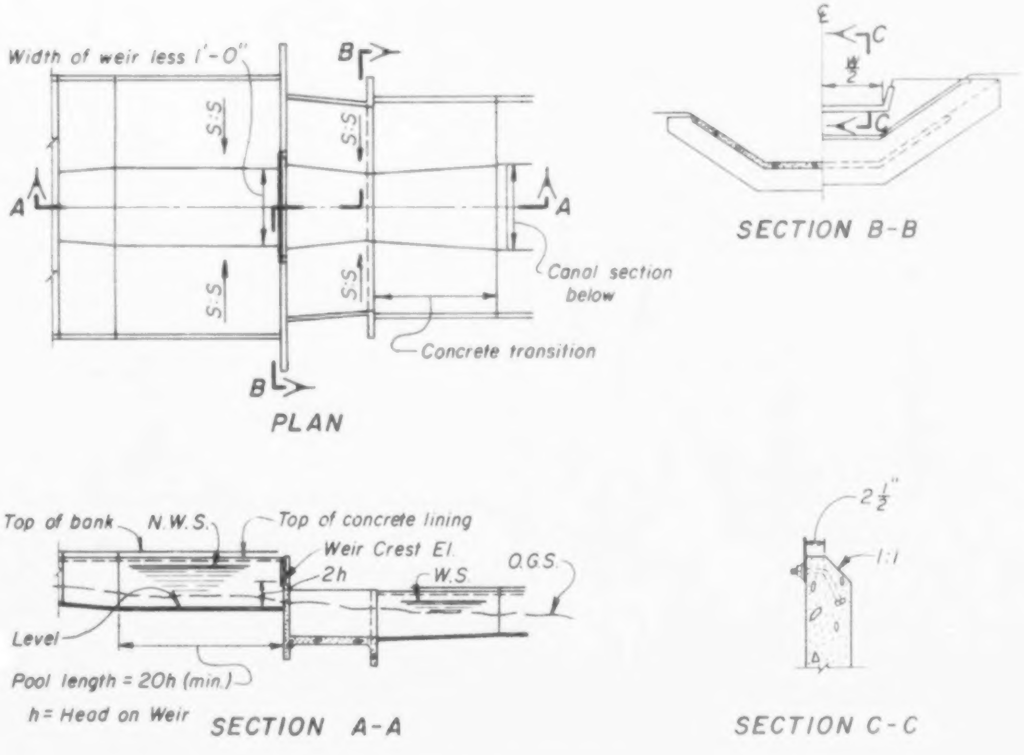
PIPE TURNOUT TO WEIR BOX

Figure 30


H_a Measuring well
$1\frac{1}{2}:1$
℄ Canal
$\frac{3}{4}$" Pipe
Throat
PLAN
N.W.S.
Original ground surface
Crest
Protection

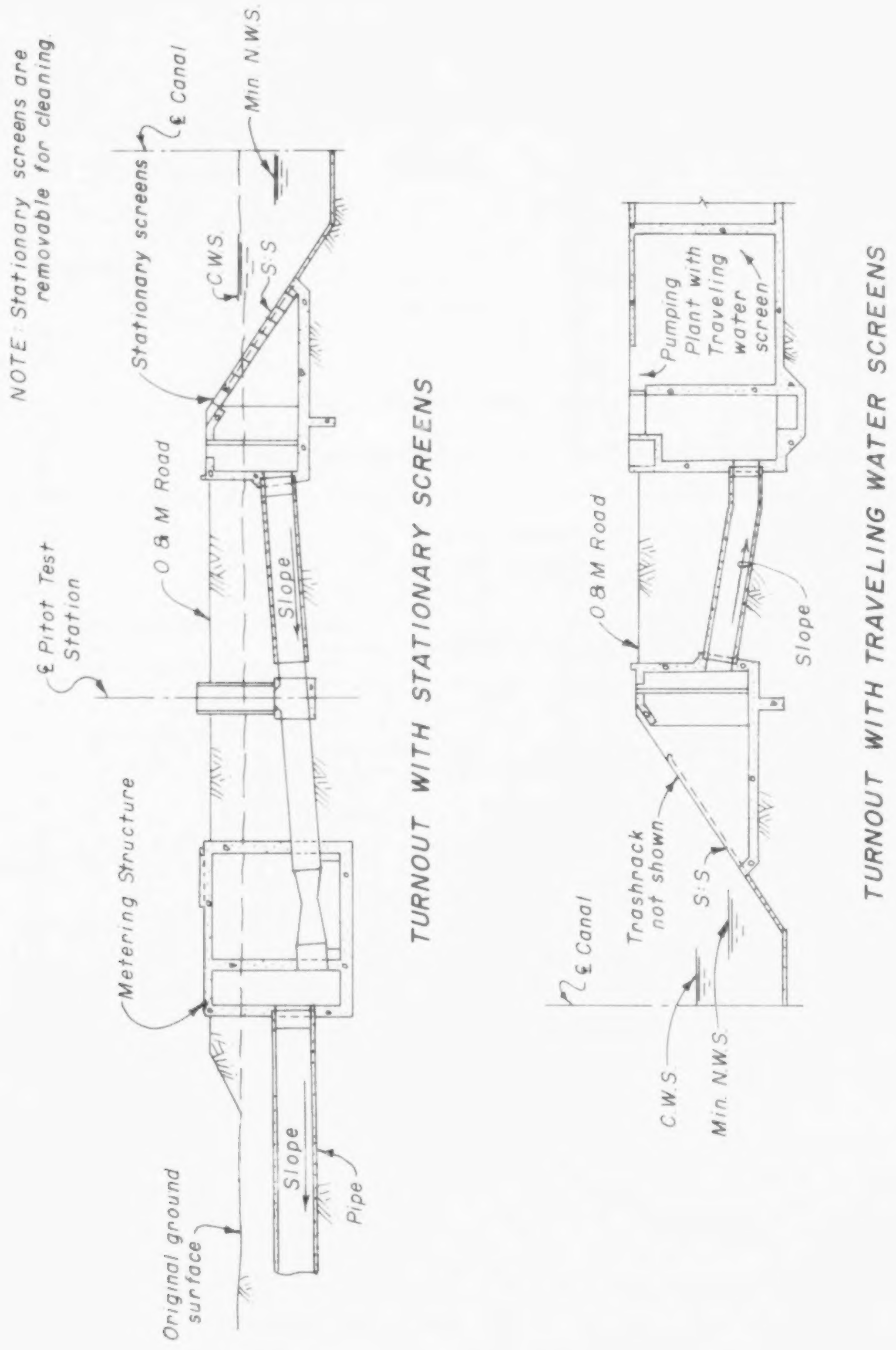
LONGITUDINAL SECTION

MODIFIED CONCRETE PARSHALL FLUME


Width of weir less 1'-0"
B
S:S
S:S
A
A
S:S
S:S
Canal section below
Concrete transition
B
PLAN
℄
C
$\frac{W}{2}$
C
SECTION B-B
Top of bank
N.W.S.
Top of concrete lining
Weir Crest El.
2h
W.S.
O.G.S.
Level
Pool length = 20h (min.)
h = Head on Weir
SECTION A-A
$2\frac{1}{2}$"
1:1
SECTION C-C

CONCRETE WEIR

Figure 31

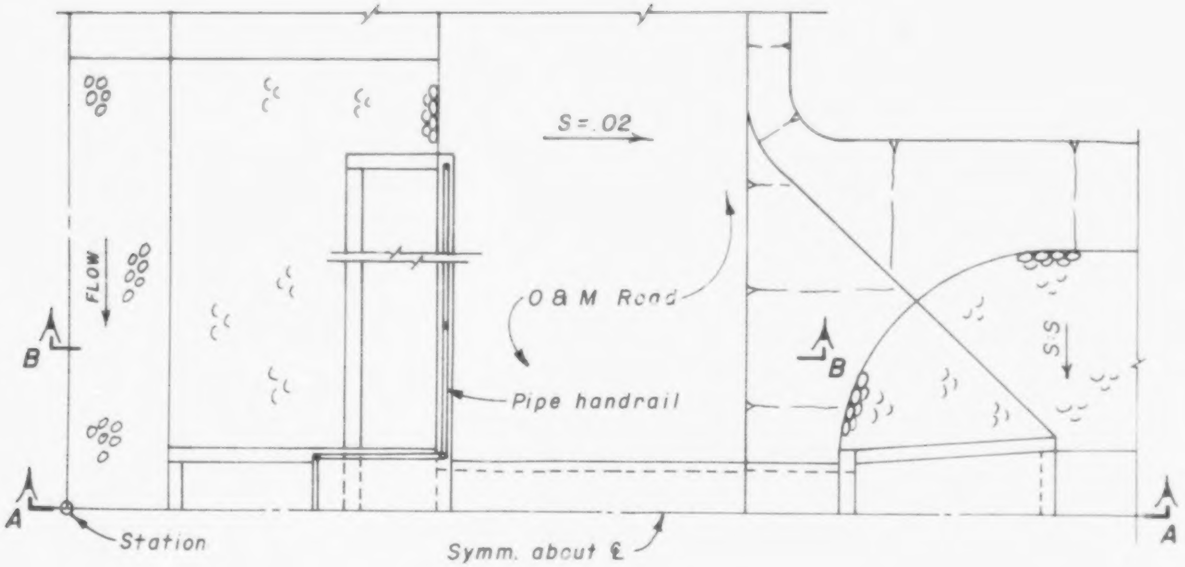

TURNOUT WITH STATIONARY SCREENS

TURNOUT WITH TRAVELING WATER SCREENS

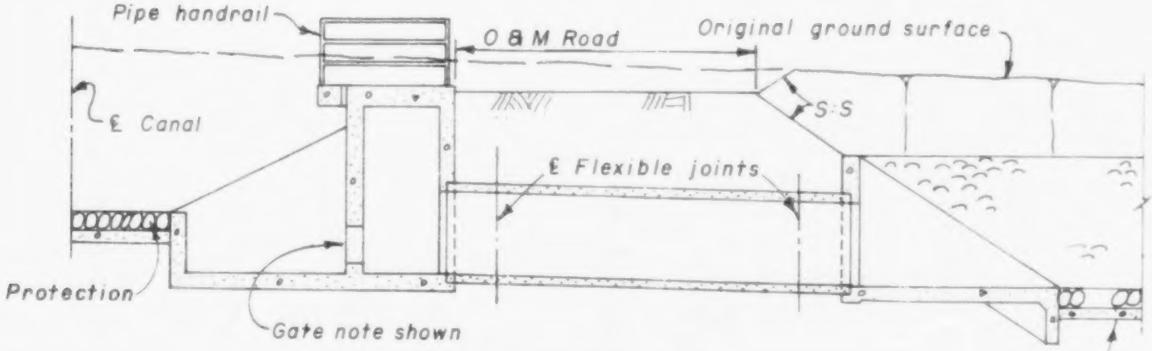
NOTE: Stationary screens are removable for cleaning.
℄ Canal
Min. N.W.S.
Stationary screens
C.W.S.
S:S
Stationary screens
O & M Road
℄ Pitot Test Station
Slope
Metering Structure
Original ground surface
Slope
Pipe
Pumping Plant with Traveling water screen
O & M Road
Slope
Trashrack not shown
S:S
℄ Canal
C.W.S.
Min. N.W.S.

TURNOUTS WITH STATIONARY SCREENS AND TRAVELING WATER SCREENS

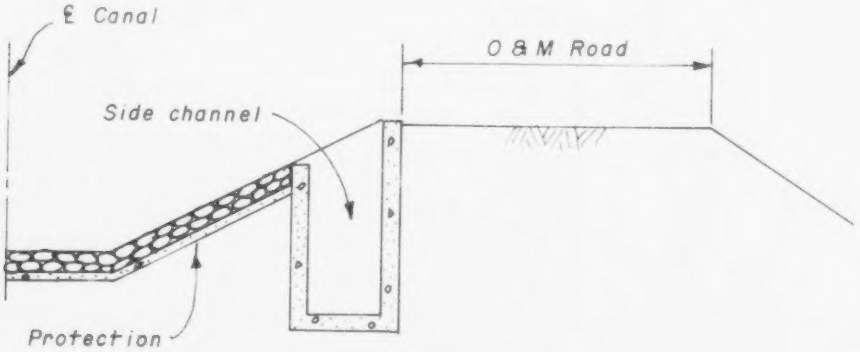
Figure 32


S = .02
FLOW
O & M Road
Pipe handrail
S:S
B
B
A
A
Station
Symm. about ℄

HALF PLAN


Pipe handrail
O & M Road
Original ground surface
℄ Canal
S:S
℄ Flexible joints
Protection
Gate note shown
Protection

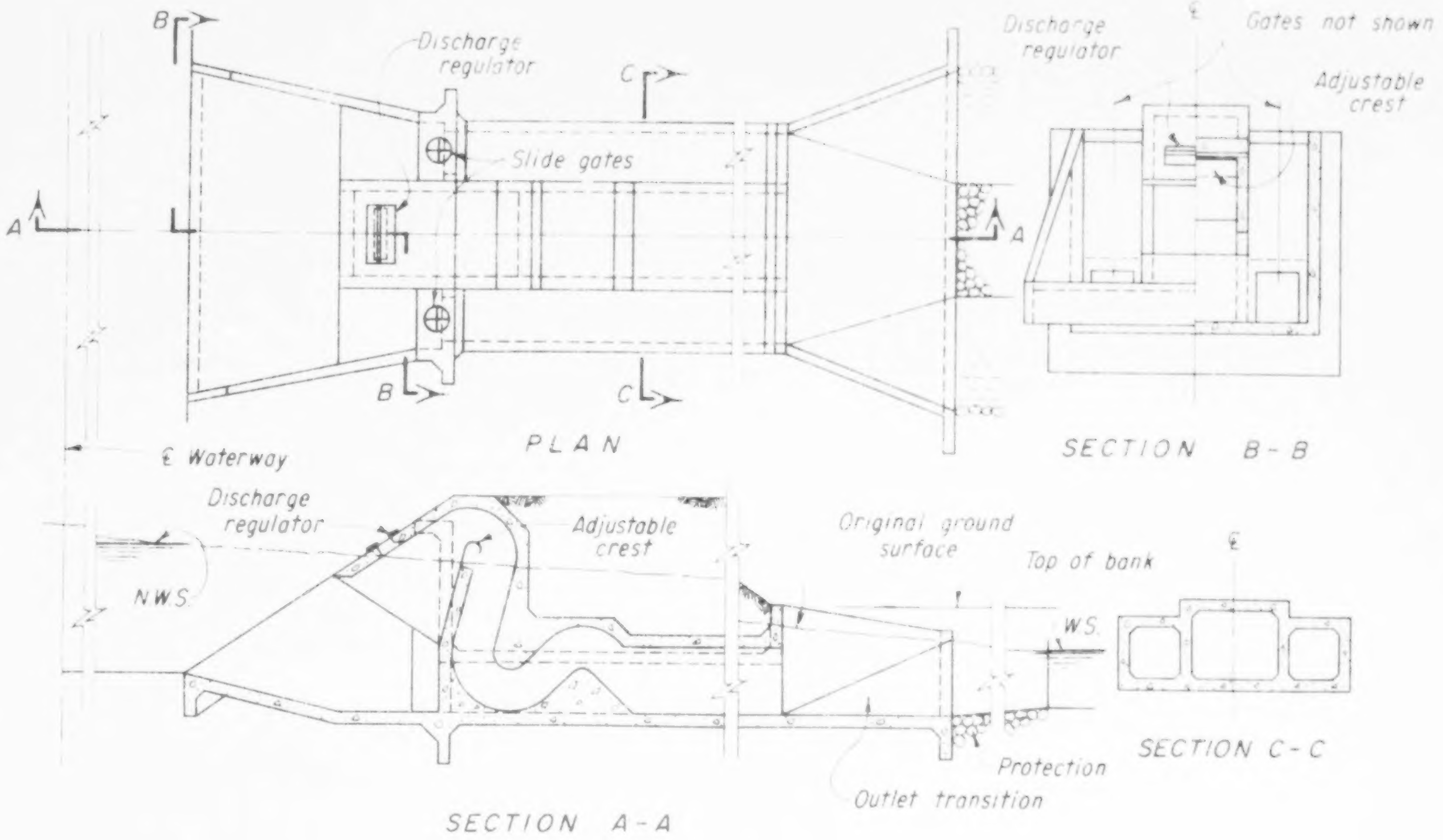
SECTION A-A


℄ Canal
O & M Road
Side channel
Protection

SECTION B-B

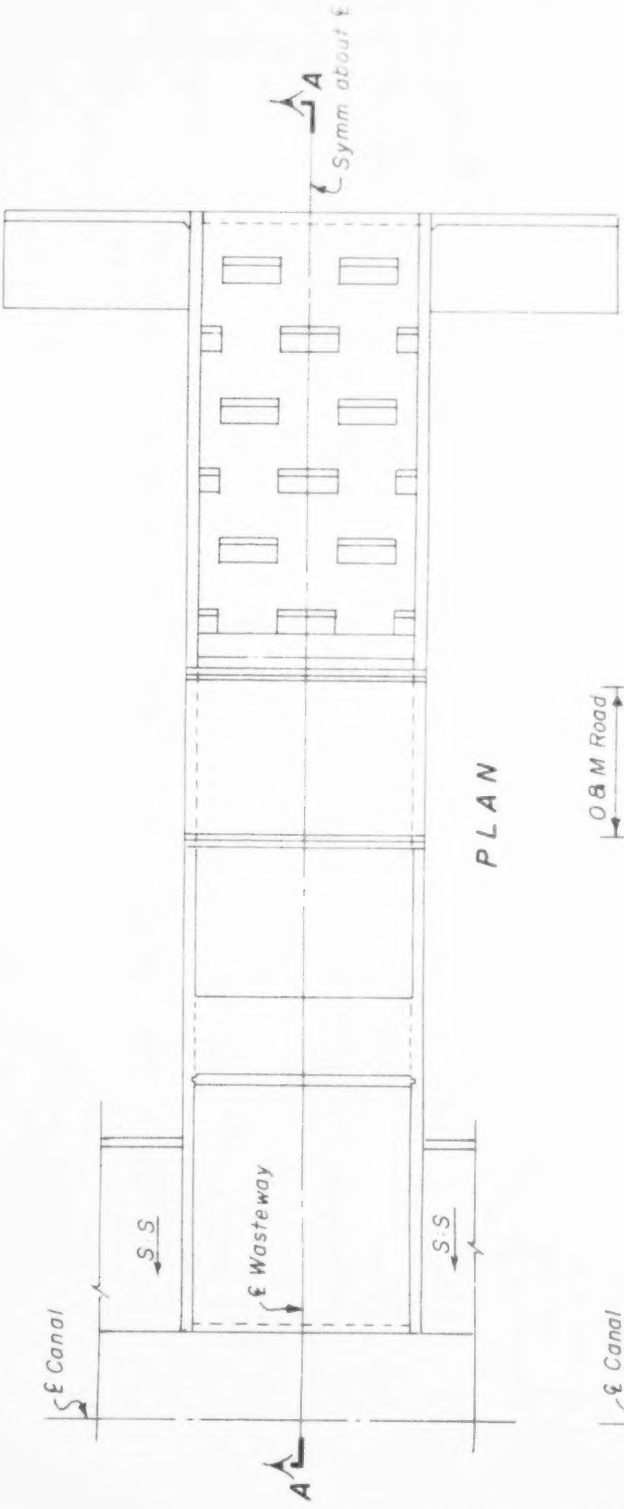
SIDE CHANNEL WASTEWAY

Figure 33

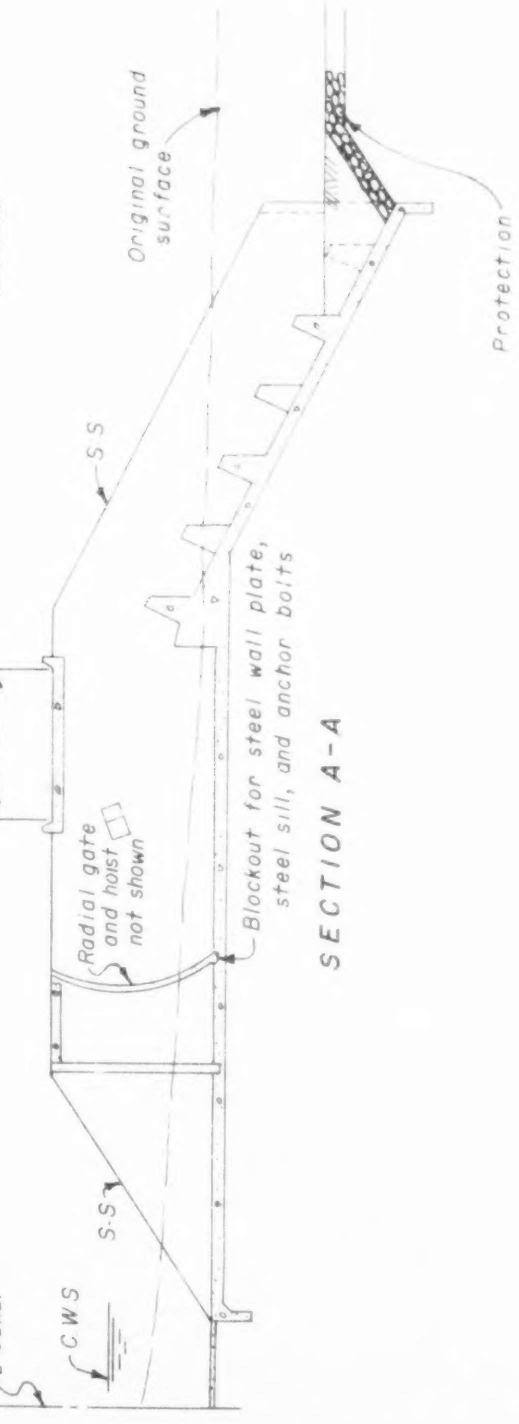

B
B
Discharge regulator
C
C
Slide gates
A
A
B
B
C
C
℄ Waterway
PLAN
Discharge regulator
℄
Gates not shown
Adjustable crest
SECTION B - B
Discharge regulator
N.W.S.
Adjustable crest
Original ground surface
Top of bank
W.S.
℄
Protection
Outlet transition
SECTION C - C
SECTION A - A

SIPHON SPILLWAY TYPE WASTEWAY WITH GATES

Figure 34

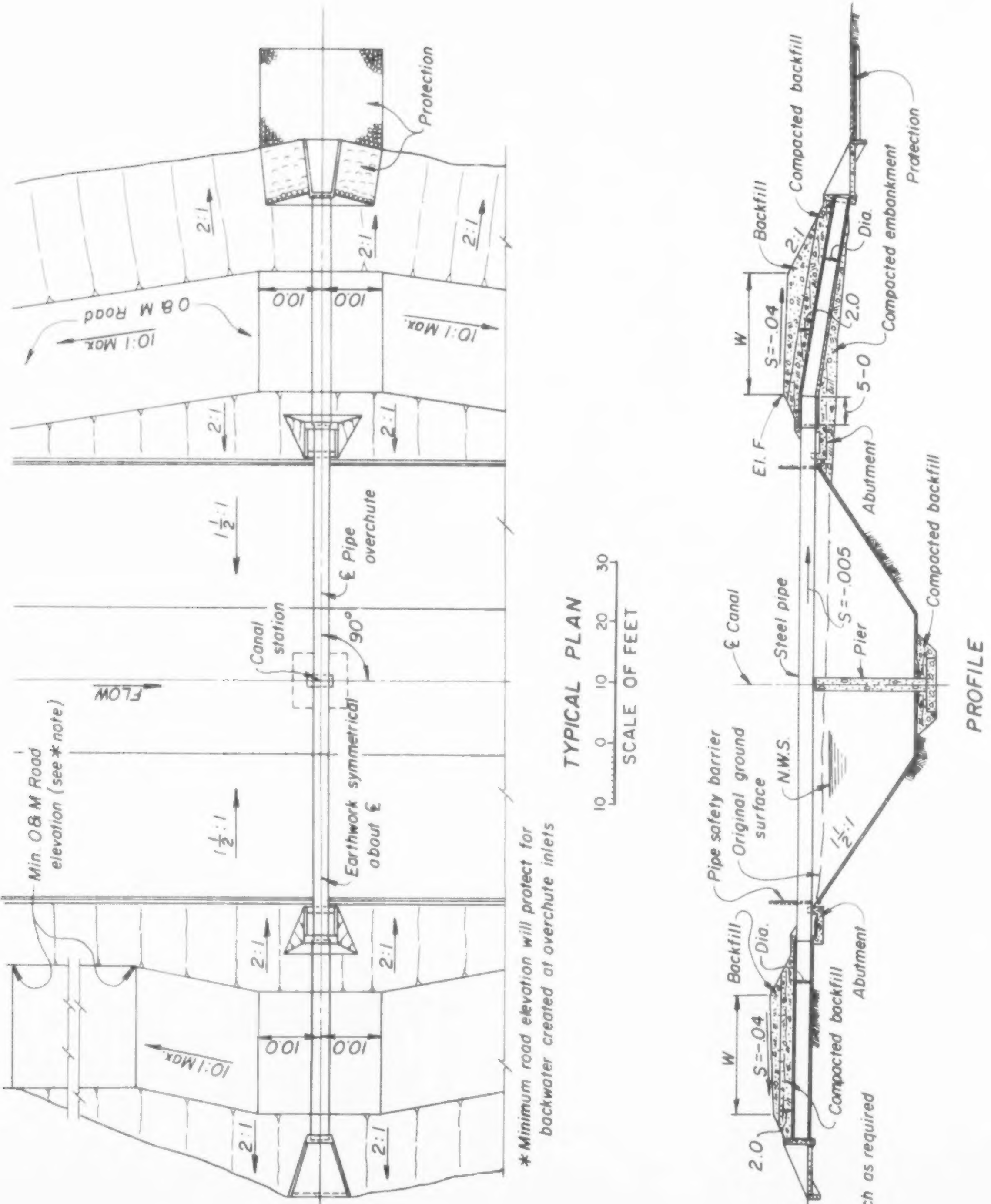

A
Symm about ℄
℄ Wasteway
S.S
S.S
℄ Canal
A

PLAN


Original ground surface
Protection
S.S
O&M Road
Radial gate and hoist not shown
Blockout for steel wall plate, steel sill, and anchor bolts
S.S
C.W.S
℄ Canal

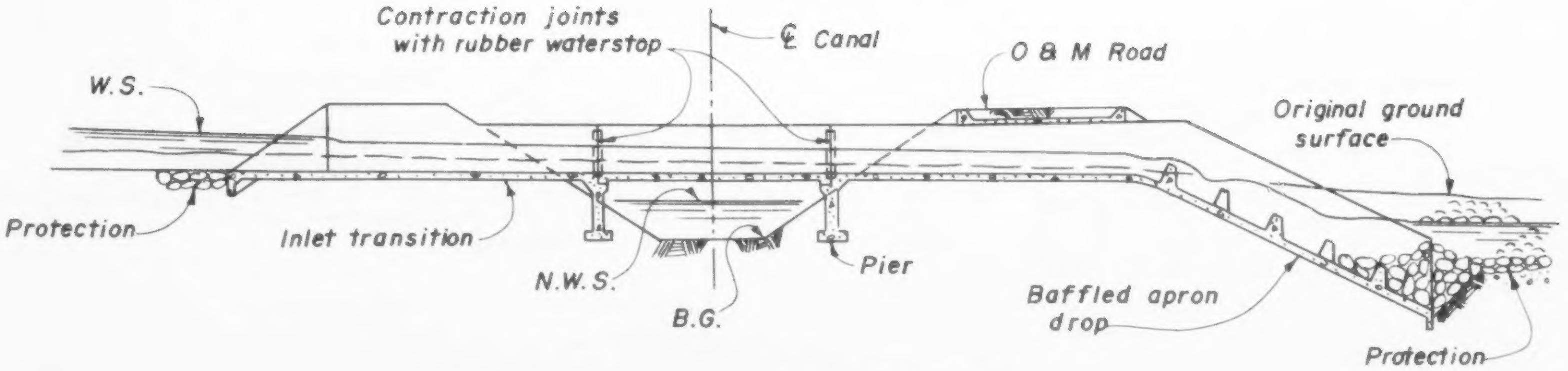
SECTION A-A

GATED WASTEWAY

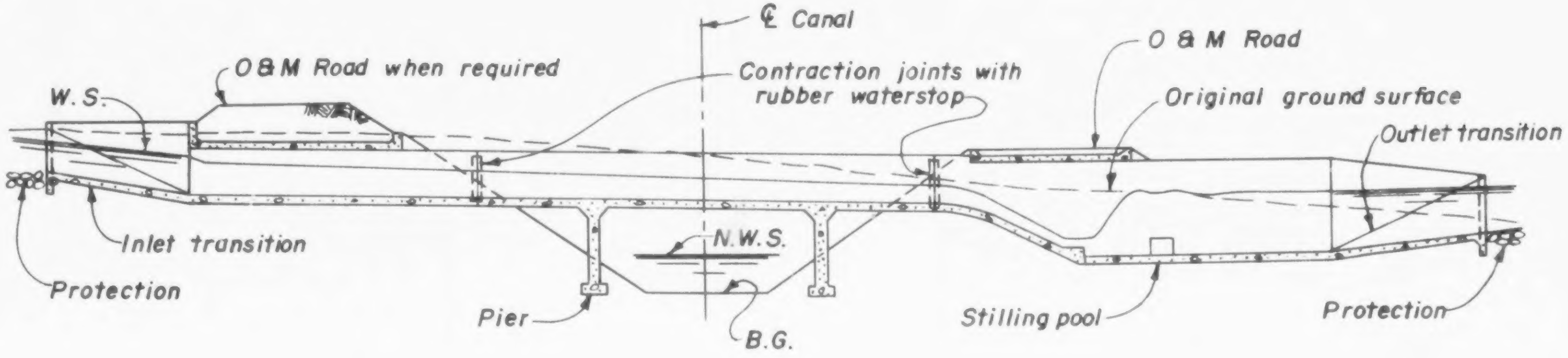

TYPICAL PLAN
Protection
2:1
2:1
2:1
O&M Road
10:1 Max.
10:1 Max.
10.0
10.0
2:1
2:1
$1\frac{1}{2}:1$
℄ Pipe overchute
Canal station
90°
FLOW
Earthwork symmetrical about ℄
$1\frac{1}{2}:1$
Min. O&M Road elevation (see * note)
2:1
2:1
10.0
10.0
10:1 Max.
2:1
2:1
* Minimum road elevation will protect for backwater created at overchute inlets
10 0 10 20 30
SCALE OF FEET
PROFILE
Compacted backfill
Protection
Backfill
Compacted embankment
Dia.
2:1
W
S=-.04
2.0
5-0
El. F
Abutment
Compacted backfill
S=-.005
Steel pipe
℄ Canal
Pier
Compacted backfill
N.W.S.
Pipe safety barrier
Original ground surface
$1\frac{1}{2}:1$
Backfill
Dia.
Abutment
Compacted backfill
W
S=-.04
Compacted backfill
2.0
Ditch as required

PIPE OVERCHUTE

Figure 36


Contraction joints
with rubber waterstop
℄ Canal
O & M Road
W.S.
Original ground
surface
Protection
Inlet transition
N.W.S.
Pier
B.G.
Baffled apron
drop
Protection

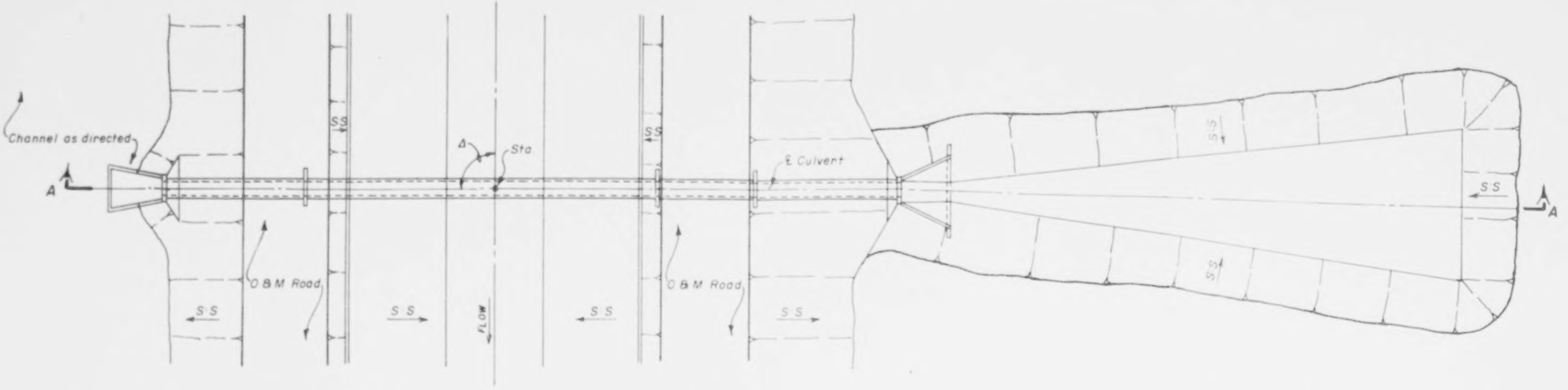
PROFILE WITH BAFFLED APRON DROP


℄ Canal
O & M Road
O&M Road when required
Contraction joints with
rubber waterstop
W.S.
Original ground surface
Outlet transition
N.W.S.
Inlet transition
Protection
Pier
B.G.
Stilling pool
Protection

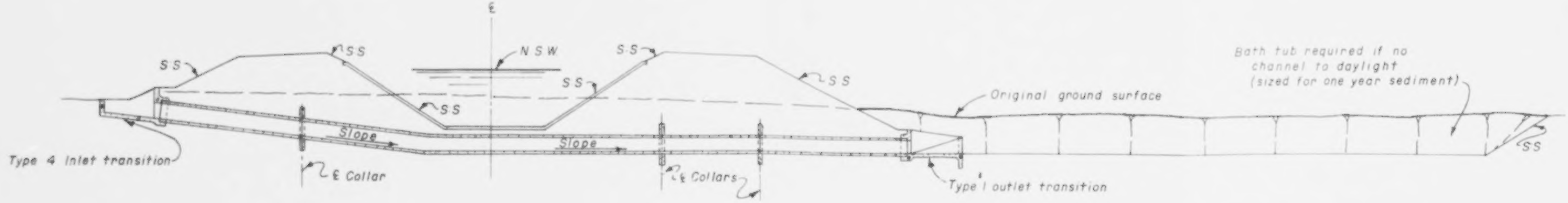
PROFILE WITH STILLING POOL

FLUME OVERCHUTES

Figure 37


Channel as directed
A
A
S S
S S
Δ
Sta.
℄ Culvert
S S
S S
O & M Road
O & M Road
S S
S S
FLOW
S S
S S
S S
S S

PLAN

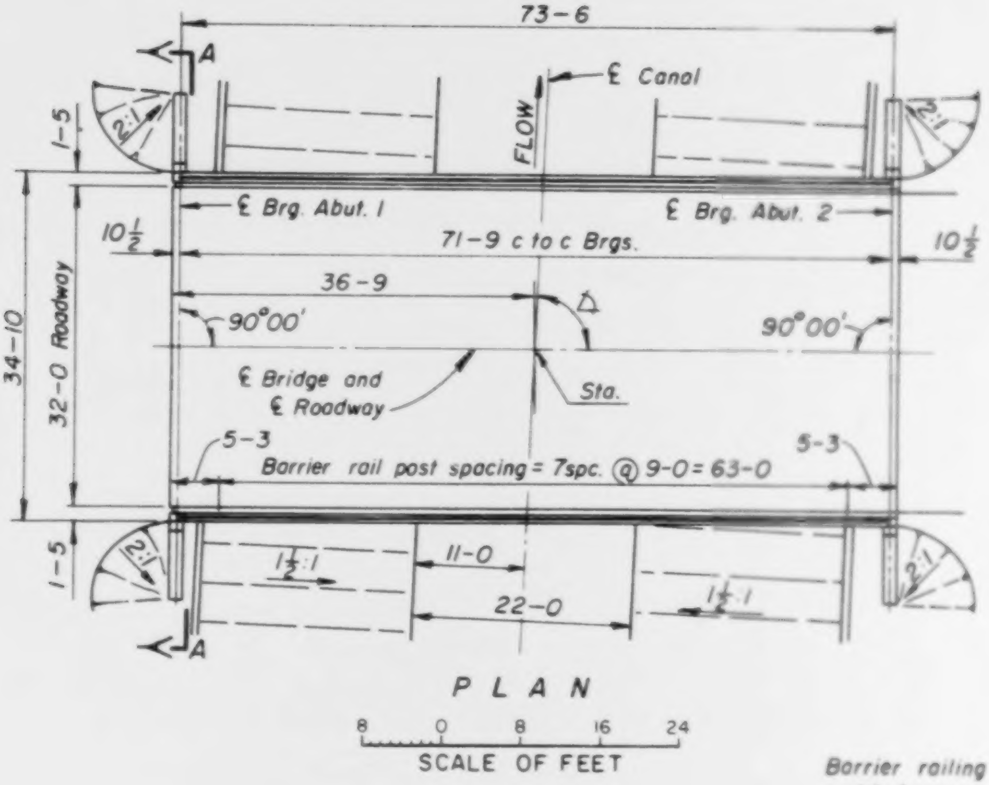

℄
S S
S S
N S W
S S
S S
S S
S S
S S
Bath tub required if no channel to daylight (sized for one year sediment)
Original ground surface
Slope
Slope
Type 4 Inlet transition
℄ Collar
℄ Collars
Type 1 outlet transition
S S

SECTION A-A

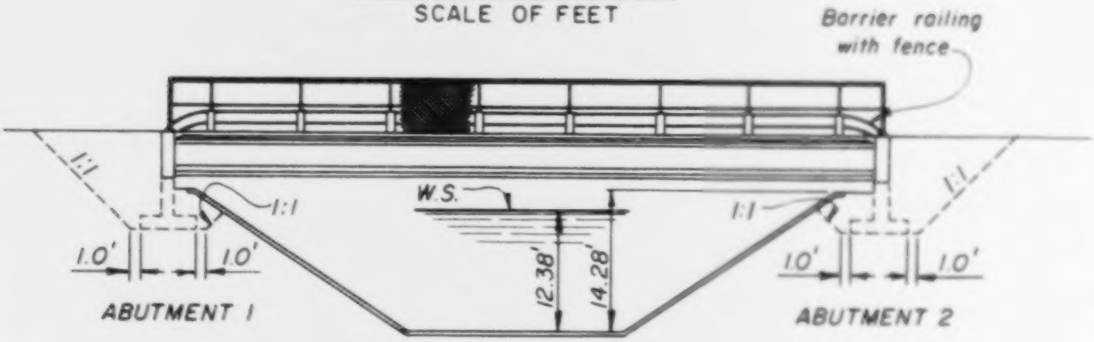
CULVERT

Figure 38

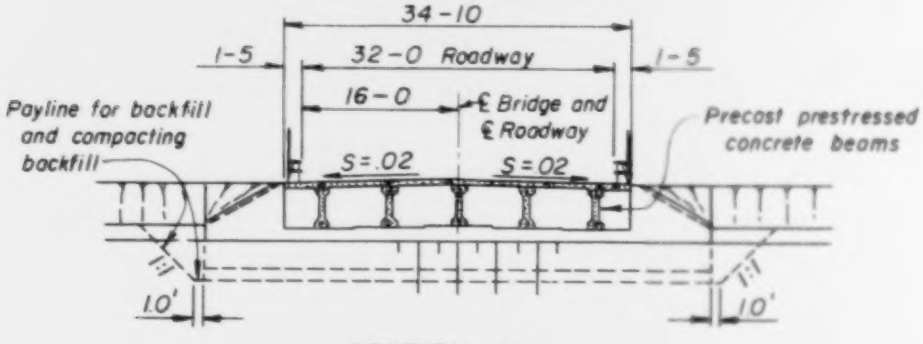
best copy available


73-6
A
℄ Canal
FLOW
2:1
1-5
℄ Brg. Abut. 1
℄ Brg. Abut. 2
10½
71-9 c to c Brgs.
10½
36-9
Δ
90°00'
90°00'
34-10
32-0 Roadway
℄ Bridge and
℄ Roadway
Sta.
5-3
5-3
Barrier rail post spacing = 7spc. @ 9-0 = 63-0
1-5
1½:1
11-0
22-0
1½:1
A
8
0
8
16
24
SCALE OF FEET

PLAN


Barrier railing with fence
1:1
W.S.
1:1
1:1
1:1
1.0'
1.0'
12.38'
14.28'
1.0'
1.0'
ABUTMENT 1
ABUTMENT 2

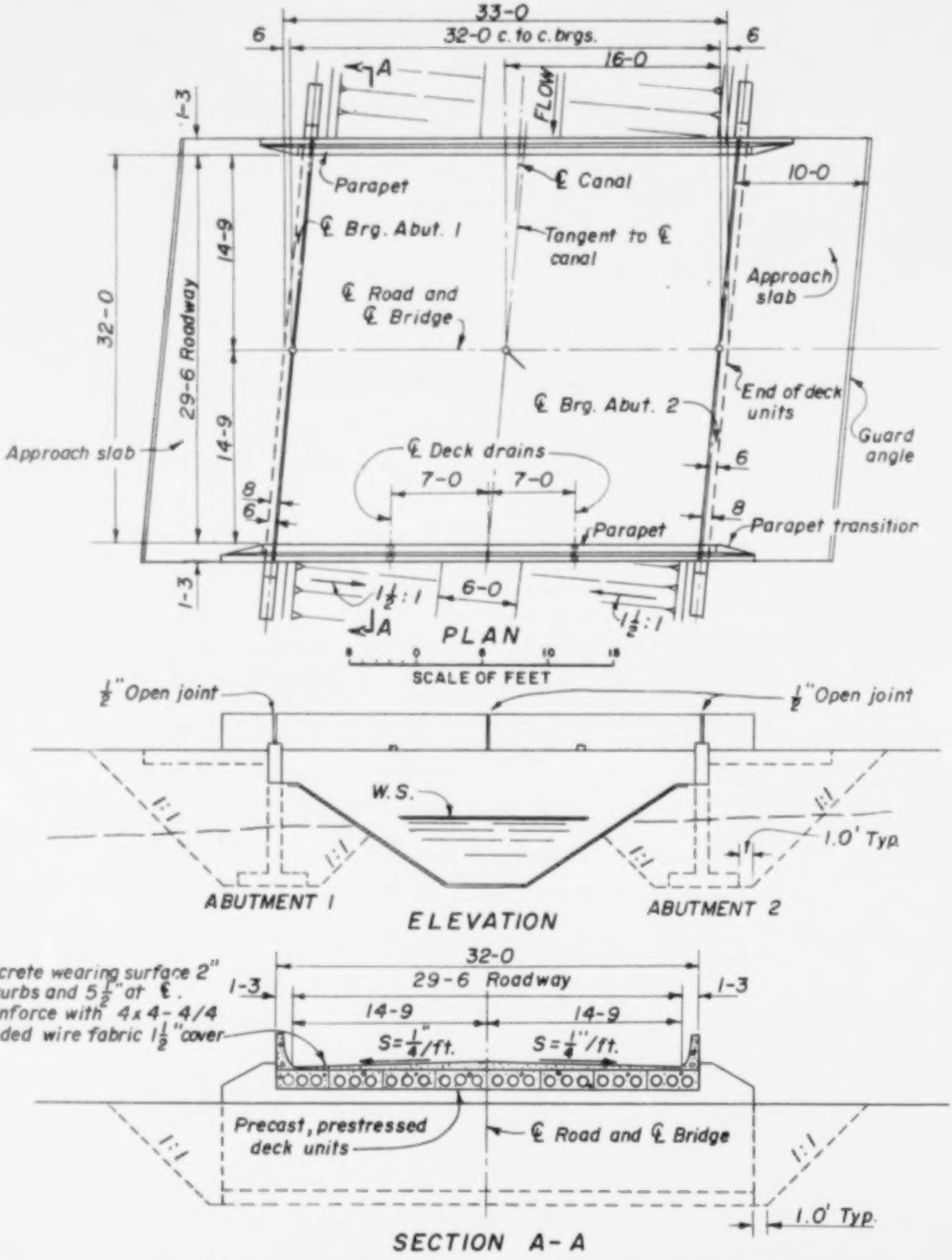
ELEVATION


34-10
1-5
32-0 Roadway
1-5
16-0
℄ Bridge and
℄ Roadway
Payline for backfill and compacting backfill
Precast prestressed concrete beams
S=.02
S=.02
1:1
1:1
1.0'
1.0'

SECTION A-A

PRECAST PRESTRESSED I-BEAM BRIDGE

Figure 39


33-0
6
32-0 c. to c. brgs.
6
16-0
A
FLOW
1-3
Parapet
℄ Canal
10-0
℄ Brg. Abut. 1
Tangent to ℄ canal
14-9
32-0
29-6 Roadway
℄ Road and ℄ Bridge
Approach slab
End of deck units
℄ Brg. Abut. 2
Guard angle
14-9
Approach slab
℄ Deck drains
6
7-0
7-0
8
6
8
Parapet
Parapet transition
1-3
1½:1
6-0
1½:1
A
PLAN
5
0
5
10
15
SCALE OF FEET
½" Open joint
½" Open joint
W.S.
1:1
1:1
1:1
1:1
1.0' Typ.
ABUTMENT 1
ABUTMENT 2
ELEVATION
32-0
29-6 Roadway
1-3
1-3
14-9
14-9
Concrete wearing surface 2" at curbs and 5½" at ℄. Reinforce with 4x4-4/4 welded wire fabric 1½" cover
S=¼"/ft.
S=¼"/ft.
Precast, prestressed deck units
℄ Road and ℄ Bridge
1:1
1:1
1.0' Typ.
SECTION A-A

PRECAST PRESTRESSED VOIDED SLAB BRIDGE

Water Systems Operation and Maintenance Workshop - 1990

FLEXIBLE CANAL LININGS AND CONCRETE JOINT SEALANTS

Session 7-1

Bill Morrison and Jay Swihart

Introduction

Conservation of our Nation's water supplies, particularly in the Western States, is becoming increasingly important as the demand continues to increase and new sources of supply become increasingly scarce. The principles of conservation require that full use be made of our natural water supplies, and the greatest results can be accomplished through reduction in the amount of water lost through seepage during transportation to the farmer's fields. It has been estimated that by lining canals, using pipe systems for the transport of water, and providing other means of seepage control on our irrigation systems, seepage losses can be reduced by an estimated 1.5 million acre-feet, which is enough water to irrigate several hundred thousand acres of land. Pipe systems will be covered in detail at this workshop by others.

Policy for Lining

The following policy for lining or placing canals and laterals in pipe was adopted in 1967:

> It is the policy of the Bureau of Reclamation in order to conserve water and to secure other benefits to consider fully the lining or placing in pipe of all constructed waterways for the conveyance and distribution of project water supplies. In those instances where the recommendations do not call for lining or pipe, full justification will be required. On unauthorized projects for which field studies are not complete, this policy shall be adopted at once. On presently authorized but unconstructed projects, and for those projects on which field investigations are completed, this policy shall be adopted in advance planning studies.

Lining Materials

Under the Bureau's OCCS (Open and Closed Conduit Systems) program, constant effort is being made to investigate, and evaluate different types of materials for canal linings. Generally, these materials can be classified under three types of linings: compacted earth, portland cement concrete, and flexible membrane, both buried and exposed.

Each proposed canal system must be given individual consideration relative to selection and usage of materials. Soil analyses, including field and laboratory testing, are necessary to provide data for design purposes, material selection, and construction control. (See session notes on "Earth Construction Practices" for additional information.)

Other factors to be considered for material selection are operating capacity and flow velocity of the canal system, structural stability requirements, water quality, safety requirements, and maintenance needs. These factors will be discussed under the individual lining types.

Compacted Earth Linings

Local materials are usually considered initially for a canal lining. Because of this, compacted earth-lined canals often represent the lowest cost type of lining. Soils used in this type lining must be of adequate impermeability, should be free from shrinkage and swelling characteristics, and should have good stability and erosion-resistant properties when placed on the canal side slopes. The criteria discussed under "Earth Construction Practices" of the session notes will assist in selecting soil materials for canal linings.

Portland Cement Concrete

This lining, placed by slip-form methods, continues to be one of the more popular linings [1]. Portland cement concrete is more resistant to erosion than most other lining materials; therefore, it is preferred for high water velocities. As a matter of fact, properly designed and constructed reinforced concrete linings will withstand velocities of any magnitude considered feasible for canals. Linings of concrete, whether reinforced or unreinforced, eliminate most weed growths with resulting improvement in flow characteristics and reduction in maintenance costs. Further, burrowing animals, which cause numerous breaks in unlined canals and in canals lined with some types of materials, cannot penetrate the concrete.

Portland cement concrete, in general, is susceptible to damage from alkali water, from alternate freezing and thawing action, and from frost heave. Consideration of locality, and the proper design and construction methods discussed under "Concrete Repair and Maintenance" of the session notes, can help to reduce the magnitude of these problems. For example, in northern climates where considerable subfreezing weather is encountered, frost heaving rather than freezing and thawing action is more of a problem. In areas where the subgrade is not free draining, provisions for adequate drainage is perhaps the most effective protection against frost heave. In the milder southwestern climates, frost heaving and alternate freezing and thawing are generally not encountered. In the intermediate localities where large daily temperature changes occur in the wintertime, freezing and thawing action can be a problem, and it should be considered in the initial selection of a lining material.

Concrete linings are also susceptible to rupture by outside hydrostatic or other pressures. They will withstand a small amount of cracking to relieve external hydrostatic pressure without significant damage.

---

[1] References listed at end of section.

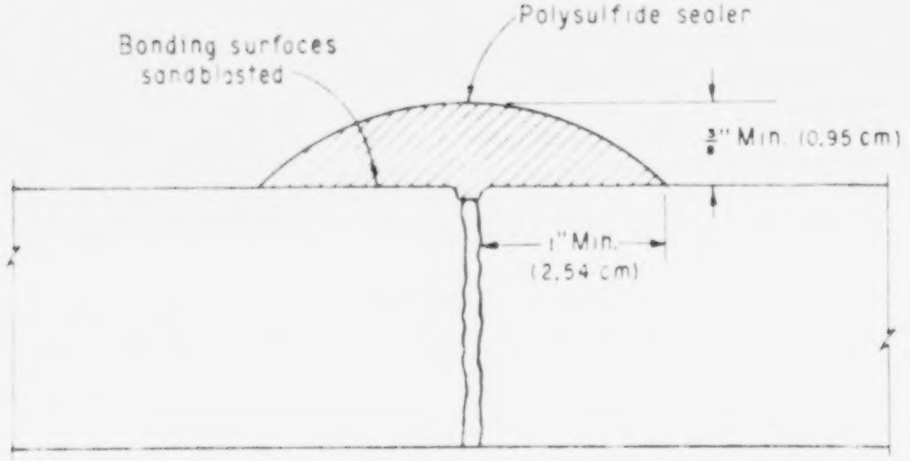
Again, drainage to relieve the outside hydrostatic pressure is generally worth the additional cost.

Due to the rigid nature of portland cement concrete, it lacks extensibility which results in cracks as contraction takes place from drying, shrinking, and temperature changes. At present, preventing such cracking is impractical; therefore, efforts are directed toward confining the cracks to selected locations by creating weakened planes, which develop into contraction joints at intervals along the canal. Cracks are generally confined to these locations, and random cracking will seldom occur if the plane weakeners are poorly spaced.

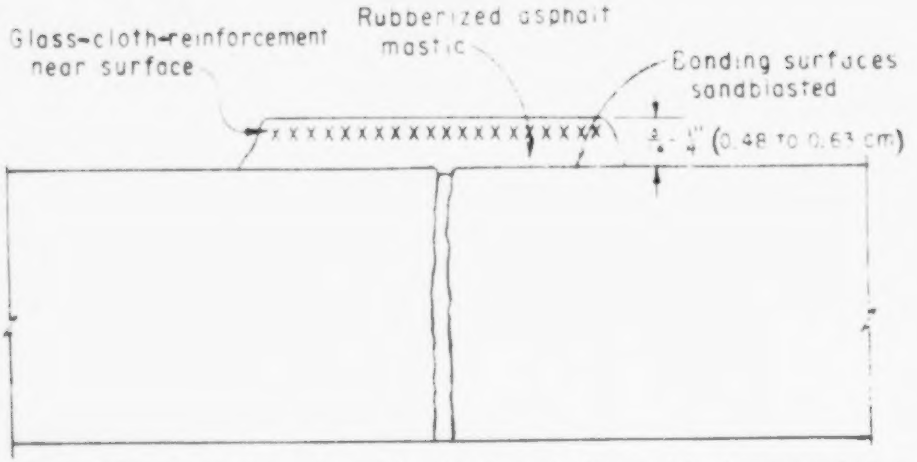
Extensive field and laboratory studies conducted during the past several years have resulted in the development of two basic systems for sealing contraction joints and two for sealing random cracks. For random crack sealing, either a molded-in-place cap strip (fig. 1a) formed from an elastomeric joint sealant or a preformed glass-cloth-reinforced asphalt mastic tape (fig. 1b) shows promise of long-term service. For contraction joint sealing, use one of the following:

a. A polyvinyl chloride plastic contraction joint-forming waterstop (fig. 2) inserted into fresh concrete linings.

b. An elastomeric sealant in both transverse and longitudinal direction (fig. 3) or transversely only in combination with a longitudinal installation of the PVC (polyvinyl chloride) strip. The elastomeric sealant is machine-extruded into hardened concrete.


Polysulfide sealer
Bonding surfaces sandblasted
3/8" Min. (0.95 cm)
1" Min. (2.54 cm)

a. Polysulfide cap seal for random cracks.


Glass-cloth-reinforcement near surface
Rubberized asphalt mastic
Bonding surfaces sandblasted
3/16 - 1/4" (0.48 to 0.63 cm)

b. Glass-cloth-reinforced, rubberized asphalt mastic tape seal for random cracks.

Figure 1. - Random crack sealing.

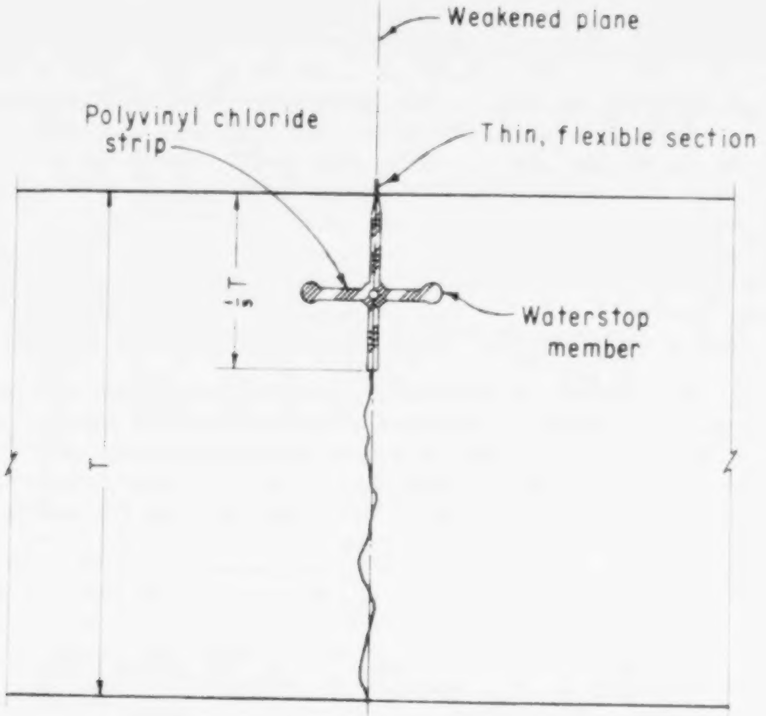

Weakened plane
Polyvinyl chloride strip
Thin, flexible section
Waterstop member
T/5
T

Figure 2. - Polyvinyl chloride sealing strip. Plastic sealing strips of the general shape shown above are inserted into the freshly placed concrete. The vertical member creates a weakened plane for crack control, and the horizontal waterstop section seals the joint.

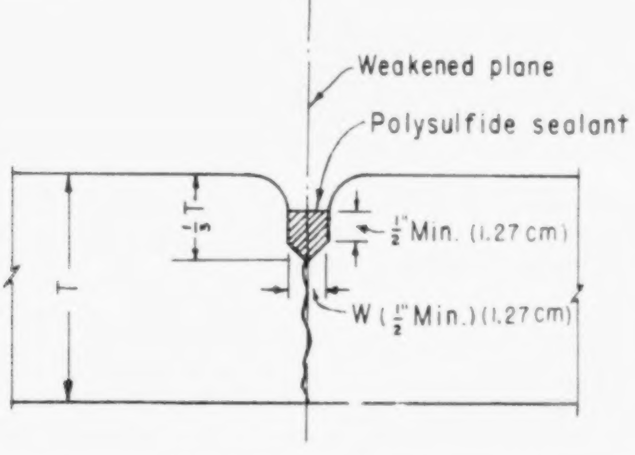

Weakened plane
Polysulfide sealant
T/5
$\frac{1}{2}$" Min. (1.27 cm)
T
W ($\frac{1}{2}$" Min.) (1.27 cm)

Figure 3. - Canal contraction joint with polysulfide sealer. Formed or tooled groove in canal lining concrete produces weakened section where contraction cracks then occur. Anticipated joint opening should not extend field-extruded polysulfide seal W (width) more than about 25 percent.

More detailed accounts of the work done on joint and crack sealing may be found in references [3] through [9].

Unreinforced concrete linings have been constructed at a significant reduction in cost, as compared with reinforced concrete linings. Velocities up to 8 feet per second are generally permissible. Unreinforced concrete linings are more susceptible to damage by hydrostatic or other pressures under the lining than are reinforced concrete linings, but not to the degree that the difference in cost might indicate. Where unexpected hydrostatic pressures are encountered under the lining, unreinforced concrete will rupture more readily than reinforced concrete, thus relieving the pressure and reducing the area of damage.

Returning to the problem of drainage, in earlier work it was generally assumed in the construction of canal linings that the lining was being placed to prevent seepage, and that the subgrade was usually relatively free draining and above the ground-water level. Unfortunately, this was not always the case, and the problem is one that has seriously damaged hard-surface-type canal linings on some projects. Typical of the damage is that shown in figure 4, where drainage from irrigated land above the canal has contributed to an increase in the water table.

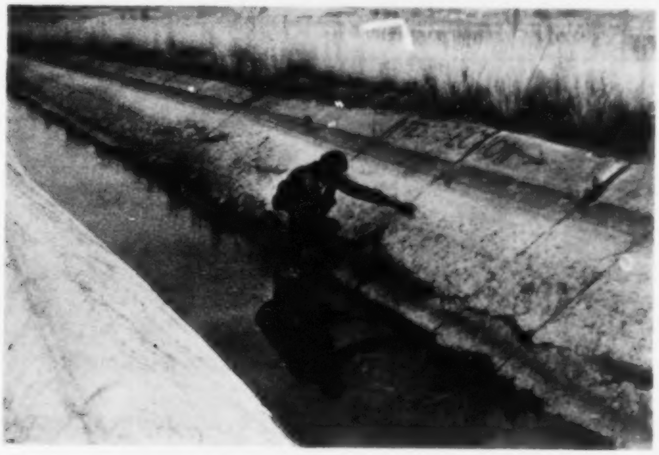


Figure 4. - Unreinforced portland cement concrete lining damaged primarily by external hydrostatic pressure due to high ground-water table. Frost action probably contributed to damage.

Proper drainage is the only means of correcting this problem. However, it may be costly and complicated. There are two common types of artificial drainage installations. One type consists of tile placed in gravel-filled trenches. These longitudinal drains are either connected to transverse cross drains which discharge the water below the canal or to pump pits, or extend through the lining and connect to outlet boxes on the floor of the canal. The outlet boxes are usually equipped with one-way flap valves which relieve any external pressure that is greater than the water pressure on the upper surface of the canal base, but prevent backflow.

A second type of drain, figure 5, consists of a permeable gravel blanket of selected material or sand and gravel pockets, drained into the canal at frequent intervals (10 to 20 feet) by flap valves in the invert. Both the tile system and the unconnected flap-valve-type drain must be encased in filter material that will prevent piping of the subgrade material into the pipe or through the valve. Material suitable for filters is described on page 305 in the Earth Manual.[10]

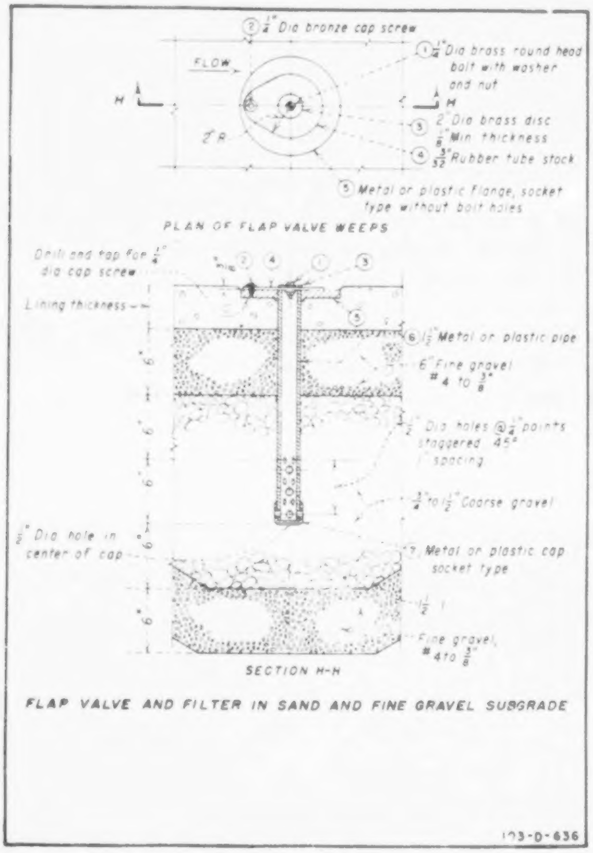

② 1/4" Dia bronze cap screw
FLOW
① 1/4" Dia brass round head bolt with washer and nut
H
H
③ 2" Dia brass disc 1/8" Min thickness
2" R
④ 3/32" Rubber tube stock
⑤ Metal or plastic flange, socket type without bolt holes
PLAN OF FLAP VALVE WEEPS
Drill and tap for 1/4" dia cap screw
Lining thickness
② ④ ① ③
⑤
⑥ 1 1/2" Metal or plastic pipe
6" Fine gravel #4 to 3/8"
1/2" Dia holes @ 1/4 points staggered 45° 1" spacing
3/4" to 1 1/2" Coarse gravel
1/2" Dia hole in center of cap
⑦ Metal or plastic cap socket type
1 1/2 : 1
Fine gravel, #4 to 3/8"
6" 6" 6" 6" 6"
SECTION H-H
FLAP VALVE AND FILTER IN SAND AND FINE GRAVEL SUBGRADE
103-D-636

Figure 5. - Flap valve installation for a canal underdrain.

A more serious problem of this nature was encountered on the Gateway Canal in the Weber Basin Project in Utah. Here, figure 6, a reach of concrete lining has been lost from the upper slope. The damaged lining is located where the canal is in quite steep sidehill locations, with the major damage confined to the lining on the upper bank. Displacement of the upper portion of the lining was most severe. The hillsides immediately above the lining in the reaches where damage has occurred were damp to wet. Water appeared to flow from permeable lenses, and similar water-bearing lenses were found when the lining against the upper bank was removed. Interceptor ditches had been constructed along the upper berm to carry off the surface water seeping from the hillside above the canal. To repair this lining and correct the condition, three methods of repair were considered:

1. Excavation of the subgrade material on the upper slope and its replacement with a heavy section of compacted earth
2. Placement of a thick, permeable gravel blanket immediately under the lining to be replaced and providing a tile drain system at the toe of the gravel blanket
3. Excavation of an interceptor trench along the upper bank which could be filled with free-draining gravel and in the bottom of which would be located a tile drain system

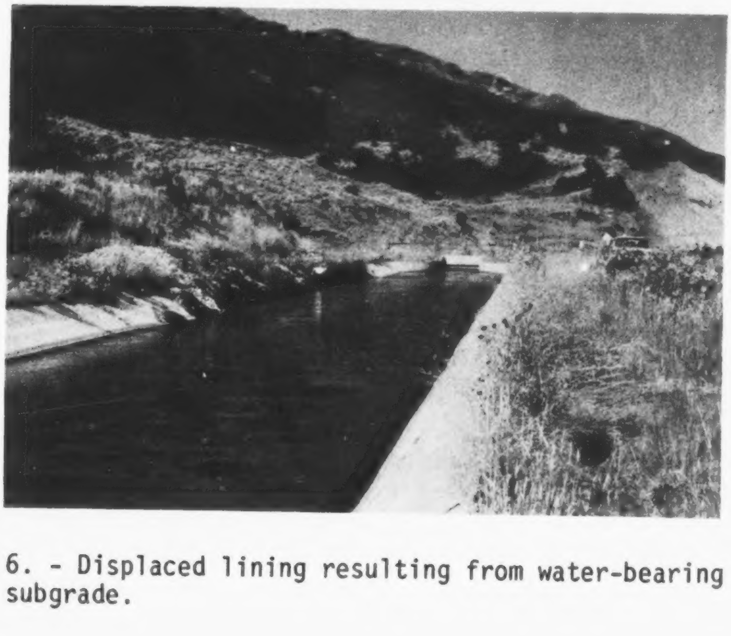


Figure 6. - Displaced lining resulting from water-bearing lenses in the subgrade.

The last method, as shown in figure 7, was selected.

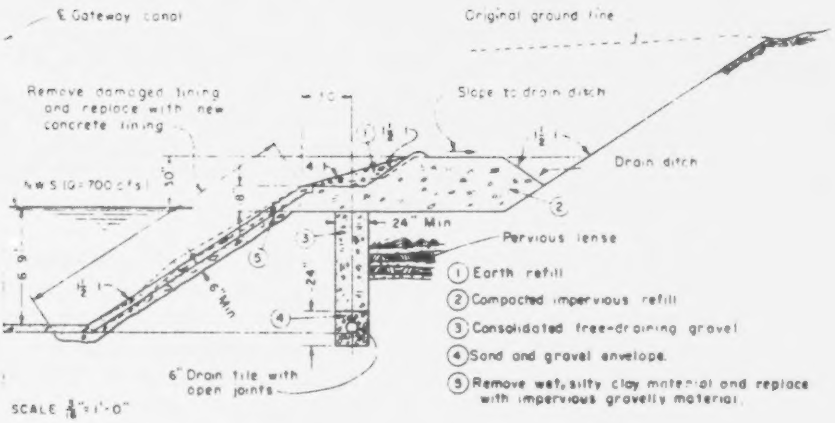

℄ Gateway canal
Original ground line
Remove damaged lining and replace with new concrete lining
10'
Slope to drain ditch
① $1\frac{1}{2}:1$
$1\frac{1}{2}:1$
Drain ditch
N.W.S. (Q=700 c.f.s.)
30"
4
8'
24" Min
③
Pervious lense
⑤
6' 9"
$1\frac{1}{2}:1$
6" Min
24"
④
②
① Earth refill
② Compacted impervious refill
③ Consolidated free-draining gravel
④ Sand and gravel envelope.
⑤ Remove wet, silty clay material and replace with impervious gravelly material.
6" Drain tile with open joints
SCALE $\frac{3}{16}$" = 1'-0"

Figure 7. - Typical detail for repair of Gateway Canal

Portland Cement Mortar

Portland cement mortar has many of the characteristics of portland cement concrete. Such mortar is usually pneumatically applied - that is, shot into place by pneumatic pressure and, when so applied, is termed "shotcrete." Shotcrete linings have the principal advantage of being more easily placed over a rougher subgrade than concrete linings and, therefore, are particularly adaptable to use in existing rock cuts where trimming to exact line and grade would be very expensive. The lining may or may not be reinforced with steel, but it has been observed that reinforcement may add many years of satisfactory service to shotcrete linings placed over earth subgrades.

Since it is seldom economical to place shotcrete linings thicker than about 2 inches, their use should be limited to small canals or to mild climates where service requirements will not be severe. Being generally constructed thinner than concrete, shotcrete linings are more readily damaged by hydrostatic pressure and by settlement, expansion, or shrinkage of the subgrade. Furthermore, the inherent difficulty in controling the thickness of the shotcrete application may result in a lining with areas where the thickness is less than specified, which would create areas of potential weakness.

Maintaining the desired thickness of shotcrete is an inspection problem and one that requires that an inspector be continually alert during lining placement. The quality of a shotcrete lining depends almost entirely upon the skill of the shotcrete gun operator, and keeping the operator aware of the thickness required is one of the most important jobs of the inspector in this type of lining work. The use of shotcrete as a resurfacing material for portland cement is discussed in the session notes under "Concrete Repair and Maintenance."

Flexible Membrane Linings

Buried Membrane Linings

This type of canal lining was the natural outgrowth of an attempt to use a relatively thin material as a barrier to the passage of water through canal subgrade soils. The most widely used materials are PVC (polyvinyl chloride) and PE (polyethylene) films. [10]

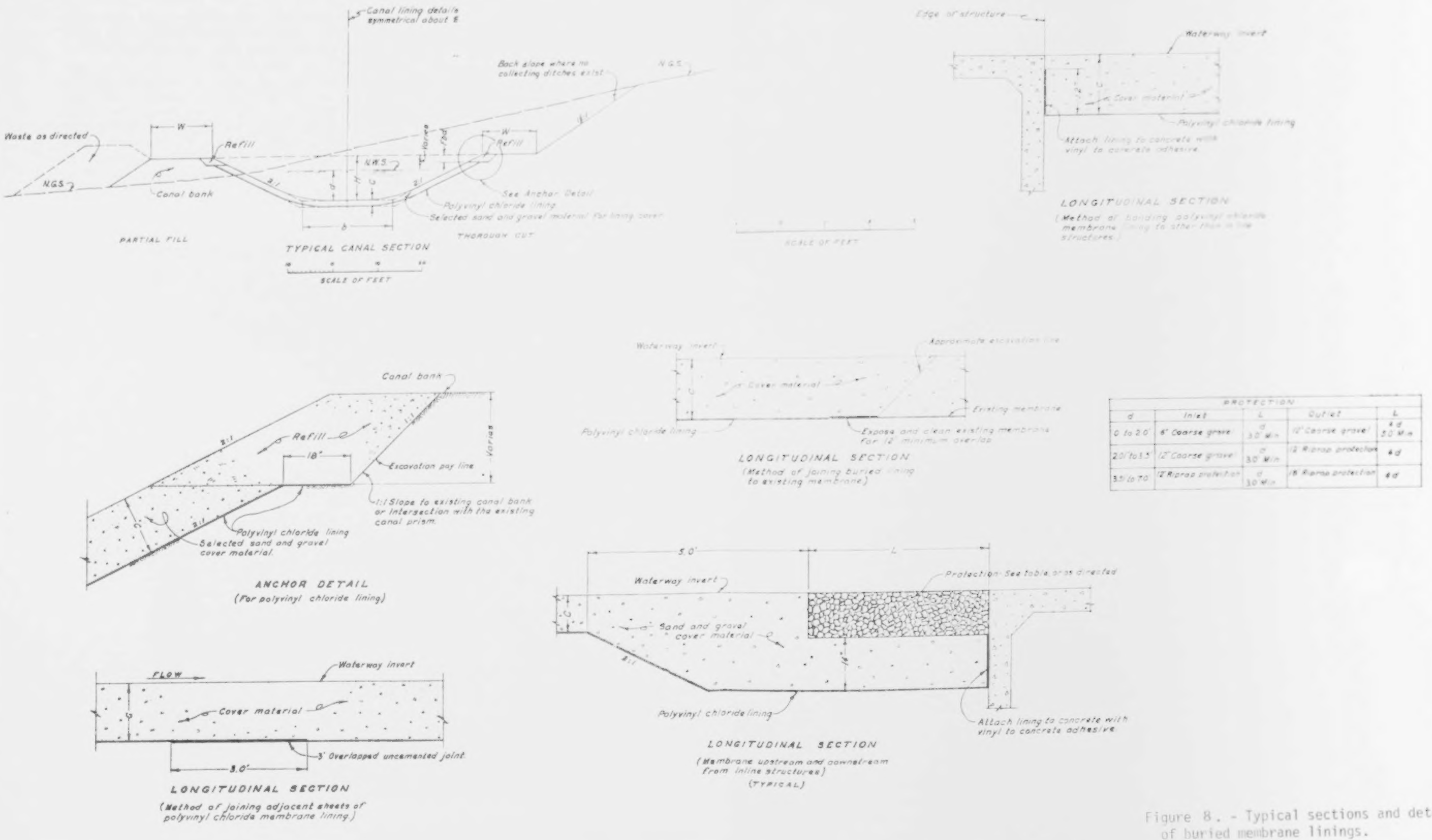
The linings are easy to install, requiring a minimum of equipment and skilled labor. They are placed on prepared canal subgrades and normally covered with a minimum of 1 foot of earth material to protect the lining from the elements and physical damage. Buried membrane linings have the advantage of being able to be placed in relatively cold or wet weather. This is important on operating projects where canal work must be done quickly in the period between irrigation seasons, often when weather would not permit placement of other types of lining materials. [11]

Where earth cover is used, the linings are restricted to canals having a low velocity flow. Maximum allowable velocity may range from 1.5 to 3.0 feet per second, depending on the type of soil-gravel cover. There are primarily two factors for a successful buried membrane lining installation: proper subgrade preparation and cover material placement.

Subgrade Preparation

Excavation and preparation of the canal subgrade are of utmost importance for the longevity and watertight integrity of the membrane. Use of a buried membrane requires that the canal be overexcavated a minimum of 1 foot to provide for placement of the protective earth cover. The side slopes are made sufficiently flat to ensure that the cover material remains on the slopes under operating conditions. The proper side slope will depend upon the type of earth cover material used, but even with the best conditions, the slope should not be steeper than 2:1. If possible, 2.5:1 to 3:1 slopes should be used. Several excellent articles have been prepared recently concerning slope stability of membrane-lined facilities. [12][13]

Bureau experience has demonstrated that a canal cross section, as shown in figure 8, produces the best design for stable cover and proper operating conditions for buried membrane linines. For the plastic membrane, it is necessary to construct anchor berms, as shown in figure 8, to secure the upper edges of the lining.


Canal lining details symmetrical about ℄
Back slope where no collecting ditches exist
N.G.S.
Waste as directed
W
Refill
N.W.S.
Varies
fbd
d
H
C
2:1
N.G.S.
Canal bank
See Anchor Detail
Polyvinyl chloride lining
Selected sand and gravel material for lining cover
b
PARTIAL FILL
THOROUGH CUT
TYPICAL CANAL SECTION
SCALE OF FEET
Edge of structure
Waterway invert
Cover material
Polyvinyl chloride lining
Attach lining to concrete with vinyl to concrete adhesive
LONGITUDINAL SECTION
(Method of bonding polyvinyl chloride membrane lining to other than in line structures)
SCALE OF FEET
Canal bank
Refill
2:1
1:1
18"
Excavation pay line
Varies
1:1 Slope to existing canal bank or intersection with the existing canal prism.
Polyvinyl chloride lining
Selected sand and gravel cover material
ANCHOR DETAIL
(For polyvinyl chloride lining)
Waterway invert
Approximate excavation line
Cover material
Existing membrane
Polyvinyl chloride lining
Expose and clean existing membrane for 12" minimum overlap
LONGITUDINAL SECTION
(Method of joining buried lining to existing membrane)
FLOW
Waterway invert
Cover material
3.0'
3' Overlapped uncemented joint
LONGITUDINAL SECTION
(Method of joining adjacent sheets of polyvinyl chloride membrane lining)
5.0'
L
Waterway invert
Protection-See table or as directed
Sand and gravel cover material
14"
Polyvinyl chloride lining
Attach lining to concrete with vinyl to concrete adhesive
LONGITUDINAL SECTION
(Membrane upstream and downstream from inline structures)
(TYPICAL)

PROTECTION				
d	Inlet	L	Outlet	L
0 to 2.0'	6" Coarse gravel	d 3.0' Min	12" Coarse gravel	4 d 5.0' Min
2.0' to 3.5'	12" Coarse gravel	d 3.0' Min	12" Riprap protection	4 d
3.5' to 7.0'	12" Riprap protection	d 3.0' Min	18" Riprap protection	4 d

Figure 8. - Typical sections and details of buried membrane linings.

After the rough excavation is completed, the subgrade is prepared to a firm, relatively smooth surface. Sharp rocks, roots, and other objects that might puncture the membrane are either removed or padded by covering with 3 or 4 inches of sand or fine-textured soil. A poor, open-graded subgrade as shown in figure 9 can result in damage to a flexible membrane. Dragging the subgrade with a heavy marine-type chain, as shown in figure 10, or an old tractor track is a rapid and effective method for smoothing the canal subgrade. For the plastic and rubber membranes, rolling may not be necessary unless compacting and smoothing of loose, coarse material are required. In larger canals, a pneumatic-tired roller can be used to roll the invert.

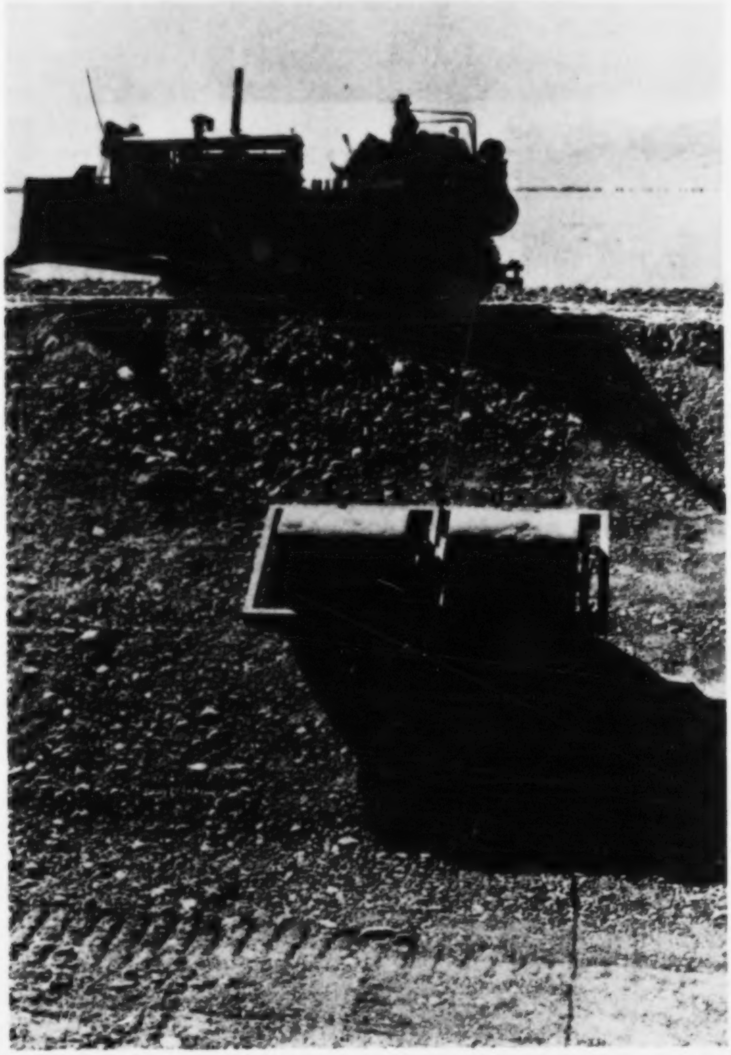


Figure 9. - Rock and nonuniform subgrade can cause severe membrane damage. Presence of finer materials in the subgrade would prevent this condition.

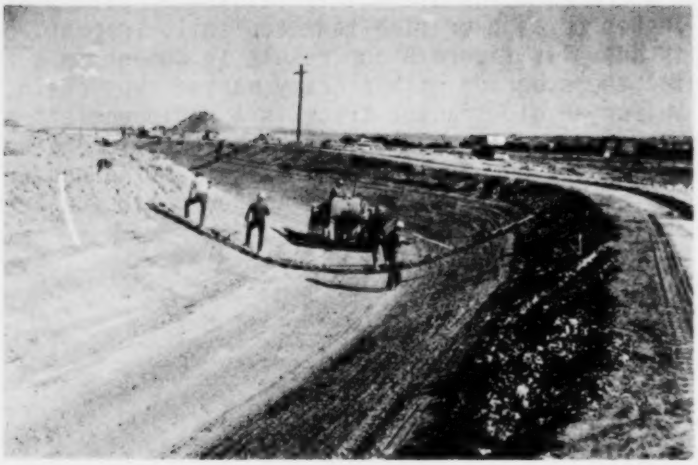


Figure 10. - Dragging the canal subgrade with tractor track in preparation for plastic membrane lining.

Weeds have not presented a problem in lining new canals because the majority of weed seeds and root systems is removed during the excavation process. However, precautionary measures are required when a membrane lining is constructed in existing canals and the subgrade is infested with cattails, tules, willows, johnson grass, nut grass, and other deep-rooted growths. The growth should be grubbed, and the soil treated with an approved sterilant. Weeds growing in the cover material are generally not expected to cause damage to the membrane linings; however, it is possible for the roots of the more vigorous plants, such as willows, to penetrate the membrane. An effective weed control program will minimize this problem.

Installation

The decision to use PVC or PE (polyethylene) plastic is made on the basis of local conditions and service requirements for each specific installation. Because the PVC is more resistant to puncture, more readily available in larger pieces (for example, widths up to 70 feet and to any length practical for handling), and more easily repaired and filed spliced with solvent-type cement, it is generally better suited than the PE for lining larger main canal systems. The PE is available in seamless widths to 70 feet and in roll lengths of 200 feet; it is spliced with a special adhedsive and tape. At the present time, low density PE is only available in sheets 10 mils thick; whereas, PVC is available in 20-mil thicknesses.

The principal advantage of the PE is its low cost. On a volume basis, PE can be purchased for about half the cost of the PVC and is thus attractive for lining small farm canals or ditches. Regardless of whether PVC or PE is used, it is important that both materials be compound by the manufacturer specifically for canal or pond lining purposes. Presently, the Bureau is specifying PVC in 20-mil thicknesses as a buried membrane lining.

Installing the plastic membrane lining in the canal prism is done simply by unfolding or unrolling the plastic and pulling it into position, as shown in figure 11. This can be accomplished rapidly with unskilled labor. The upper edges of the plastic lining are buried in anchor trenches excavated along the berms or slopes of the canal. Membrane is placed in the canal in a slack condition so that the weight of the earth cover will not cause severe stressing.

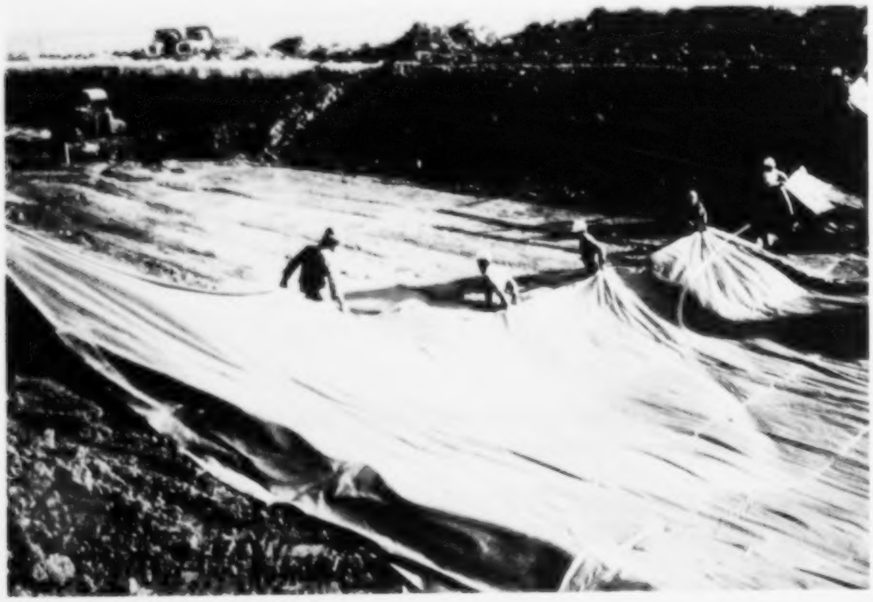


Figure 11. - Installing 20-mil PVC plastic lining on Pilot Canal, Riverton, Wyoming. Edges of lining were buried in anchor trenches by hand.

Until the cover material can be placed, the plastic lining should be held in place by sandbags, small random-placed earth mounds, or other satisfactory means. As an example, discarded rubber tires have been used as temporary weights. Canal maintenance is ordinarily limited to keeping the protective cover in place over the flexible membrane lining. Figure 12 shows placement of the lining up the side slopes and on the anchor trench.

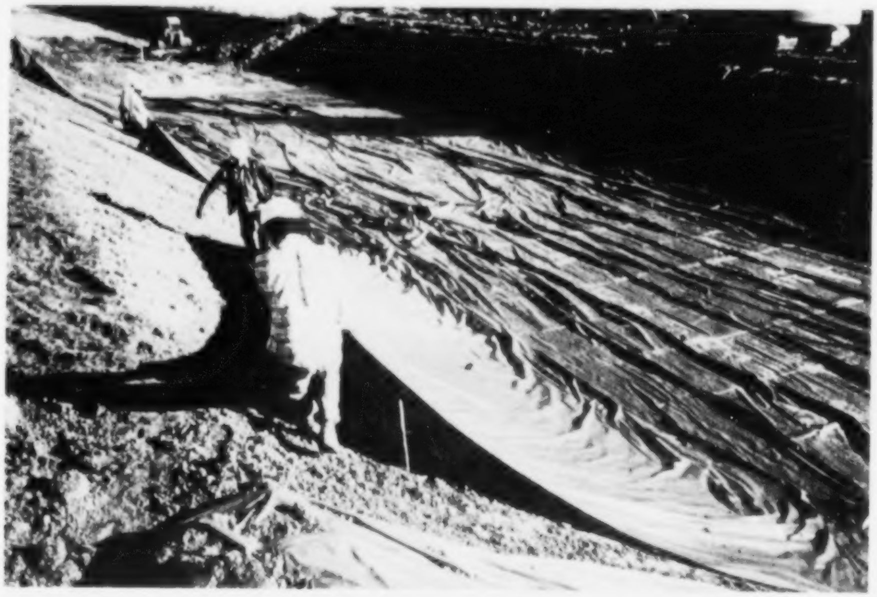


Figure 12. - Placing lining material on side slopes and anchor trench.

Cost data on a recent 20-mil-thick PVC membrane lining installation are given in table 1.

Table 1. - Cost of 20-mil PVC lining for Pilot Canal, Riverton Unit, Wyoming, USBR Specifications 63-C0031, June 1981

Hydraulic properties: Q = 756 ft^3/s; V = 2.17 ft/s
side slopes = 2:1;
base = 36 feet;
water depth = 7.19 feet
Length: 8,449 feet
Quantity: 78,200 yd^2

Table 1 - continued

Work description	Quantity and unit	Engineering estimate per unit	Bid per unit	Cost
Excavation	64,000 yd^3	2.00	1.72	$110,080
Preparing subgrade for membrane lining	78,200 yd^2	1.10	0.516	40,351
Furnishing and placing PVC (20 mils thick)	78,200 yd^2	2.10	1.118	87,427
Placing earth cover (9 inches thick)	18,300 yd^3	2.00	1.72	31,476
Furnishing and placing sand and gravel cover (9 inches thick)	19,500 yd^3	6.00	5.16	100,620
Refill	2,900 yd^3	2.00	4.30	12,470
Furnishing and placing riprap	150 yd^3	25.00	12.90	1,935
Total cost ($4.92 yd^2)				$384,359

A recently completed study [10] indicated that PVC linings are performing their intended function of seepage control even after 19 years of service.

Cover Material

The protective earth and gravel cover plays an important role in the longevity of a buried membrane lining. The soil material used should be obtained from approved areas. The inclusion of rocks, boulders, vegetable matter, brush, large roots, and other objectionable foreign matter should not be permitted. A recent Bureau study [14] reports on the performance characteristics and observed stability of 19 selected canal reaches.

The cover material should be such that it will be free draining, erosion resistant, and stable on the side slopes. With some installations, it may be possible to use the material excacated from the canal or a combination of this material and a gravel blanket. For best serviceability, the cover should be a gravelly material within gradation limits shown in figure 13.

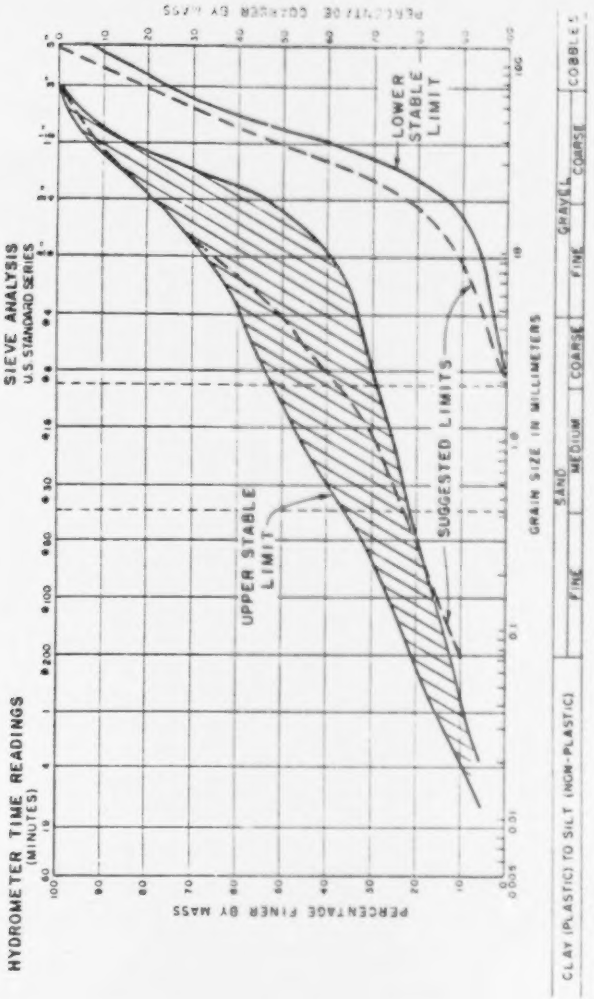


Figure 13. - Gradation ranges for good to poor performance of coarse soil cover layers with suggested new limits. [14]

Proper placement of the cover material is important to avoid possible puncturing, tearing, or stressing of the membrane. Draglines, backhoes, conveyors, or other approved equipment may be used for placing the cover material as shown in figure 14. Placing of the cover material should be conducted in such a manner that the lining will not be displaced or injured. Also, the cover material should not be dropped on the liner from heights in excess of 3 feet. In addition, it is recommended that the side slopes be covered first. It should be pointed out that some consolidation of the cover material is expected to occur soon after the canal or lateral is placed in service. This will often involve some minor repairs to the cover material.

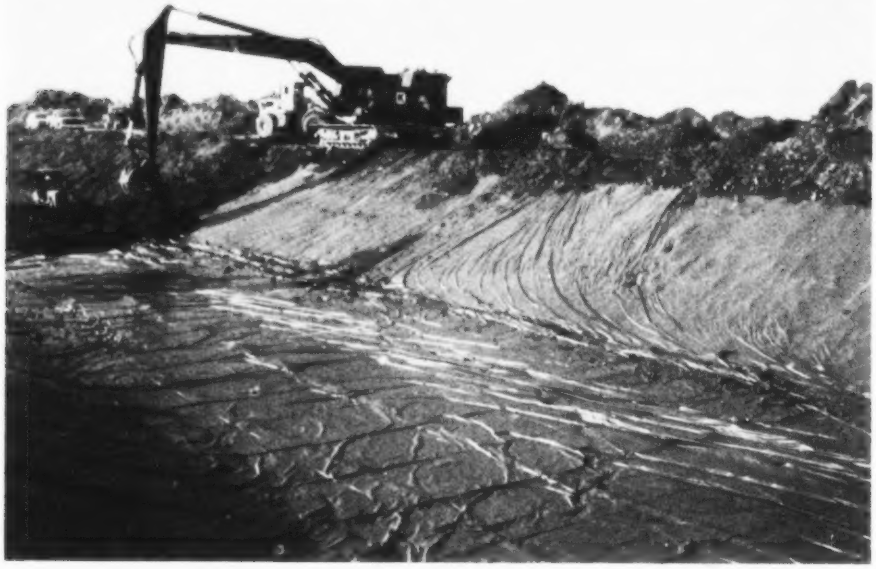


Figure 14. - Placing sand and gravel cover material on bottom of canal prism after laying 20-mil-thick PVC sheeting.

Exposed Membrane Linings

Relatively thin flexible membranes of plastic or rubber (elastomers) and other synthetic materials placed on the inside surface of a canal and exposed to the elements and action of the flowing water are commonly designated as exposed membranes. These linings generally have a thickness of one-tenth inch or less, as compared to the thicker linings of portland cement concrete, etc., which also are exposed to the wear and tear of the elements and flowing water. Weathering, vandalism, erosion, animal traffic, weed penetration, and cost for the more durable and thicker polymer materials must be taken into consideration. Consequently, use of exposed membranes on canals has been almost entirely experimental by the Bureau although used by other foreign and domestic agencies.

Installation practices for exposed membranes are much the same as for the buried membrane linings with the exception of better subgrade preparation and a deeper anchor trench at the top of the slope. The exposed membranes are exposed on the side slopes of the canal only and buried in the invert for stability and maintenance purposes.

The most common flexible membrane materials available for exposed lining work are:

1. Butyl rubber (isobutylene-isoprene)
2. EPDM (ethylene propylene diene monomer)
3. Hypalon (chlorosulfonated polyethylene)
4. CPE (chlorinated polyethylene)
5. HDPE (high density polyethylene)
6. HDA (high density polyethylene alloy)

The membrane sheet materials are supplied either unreinforced or with a reinforcing fabric laminated within the sheet to increase tensile and tear strength and improve puncture resistance.

Where the possibility of damage to exposed membranes is minimal, they are suited for lining sections of old, leaky, concrete canals. This is especially so where the canal can be taken out of service for only a short time, thus making standard repairs an impossibility. For example, in 1964, a 400-foot section of concrete lining in the Contra Costa Canal, Central Valley Project, was lined with a 1/32-inch-thick butyl-rubber sheeting. The canal has a Q of 155 cubic feet per second and a V of 2.74 feet per second. With some repairs, this lining is performing satisfactorily.

Exposed flexible membranes have also found limited use in Bureau rehabilitation work. In November 1975, approximately 16,000 square feet of Hypalon were installed on an experimental basis in the Lambrecht Reservoir, Solano Irrigation District, California. The lining material was furnished and installed by the manufacturer. Lambrecht Reservoir was selected as a test site because the original lining, an asphalt prefabricated membrane, has deteriorated badly as a result of wave action and exposure to the sun. The reservoir, constructed in 1961, has a lining surface area of approximately 12,000 square feet, a depth of 8 feet, and a maximum volume of 1.8 acre-feet. It is an irregular teardrop shape with a maximum length and width at the berm of approximately 165 and 120 feet, respectively.

The lining material was shipped to the site in two panels which has been factory seamed, folded, and rolled into a compact package for handling. To join the panels in the field, a seam adhesive consisting of a Hypalon-base compound dissolved in trichloroethylene was used. The seaming process was accelerated by applying heat to the seam with a small hot-air gun. This heat application is not necessary when the ambient temperature is about 70 °F. This installation has been monitored periodically by the Solano Irrigation District and Bureau to determine the effectiveness of Hypalon in rehabilitation work. To date, this lining has performed its intended function very well.

The commerically available materials for use as exposed membranes present viable alternatives to both concrete and buried plastic linings. In particular, when used in place of traditional buried PVC or PE, the following advantages can be realized.

1. Canal side slopes can be increased to 1-1/2:1 or even 1:1 with subsequent cost savings in reduced excavation and right-of-way.

2. Substantial savings, associated with borrow areas, hauling and placement, are realized.

3. Undetected damage to a membrane lining is minimized.

4. Damage can be detected early and repaired by O&M personnel.

5. A greater volume of user water can be delivered for a given cross section as compared to an earth-lined canal with possible reduction in dimensions of the required canal prism and subsequent cost reduction in excavation and right-of-way.

6. Replacement of an exposed membrane liner is easier and can result in cost savings at end of design life.

Additional cost savings could also be realized by using a geotextile (synthetic fabric) under the exposed membrane, resulting in reduced subgrade preparation. The geotextile would provide a smooth surface for membrane installation and seaming. Also, the geotextile can increase puncture resistance for thin, unreinforced membranes.

Disadvantages and potential problem areas are always present with any system and should be studied thoroughly. Exposed polymeric canal (exposed side slopes only) membrane linings can be subjected to: wind and water forces, possibility of vandalism, possibility of animal damage, minor ultraviolet surface degradation over time, and abrasion.

Bottom-only lining

In some special situations soil conditions are such that seepage is primarily vertical. Under such conditions, a bottom-only lining may be attractive. Since a completely watertight canal is not necessary, the small amount of seepage from the side slopes can be tolerated if the cost advantage are sufficient. Soil conditions which exhibit this seepage pattern are found in the loessal soils in some areas of Kansas and Nebraska. Field studies were conducted at three locations by the Nebraska-Kansas Projects Office, Grand Island, Nebraska. For these studies 10-mil PVC was installed only in the canal insert. Another advantage of the this type of lining system is that it eliminates the problem of cover stability on side slopes. Results of the field studies [15] are listed below.

Location	Canal Size*		Seepage loss $ft^3/ft^2/day$		Percent reduction
	Q	B	Before lining	After lining	
Farwell Main Canal	290	18	0.56	0.27	52
Franklin Canal	230	14	1.01	0.45	55
Upper Meeker Canal	284	16	1.13	0.56	50

*Q denotes flow in ft^3/s.
B denotes bottom width in feet.

Seepage Measurements

When lined during original construction, the canal lining is not placed only where soil logs indicate the presence of impermeable subgrade materials or where such materials are encountered during construction. Currently accepted methods of measuring the watertightness of canal sections are limited to preconstruction permeability tests, usually made with a well permeameter, figures 15 and 16, or by excavation of a pond along the proposed line of the canal. In sections lined after the canal has been placed in service, the lining requirement may be readily apparent, but more often it is necessary to resort to field tests to determine the source of the leakage. The determination of the amount of seepage from existing canals is generally investigated by ponding tests, inflow-outflows tests, and seepage meters. Each method has advantages and limitations, and no single method is adaptable to all conditions encountered in the field.

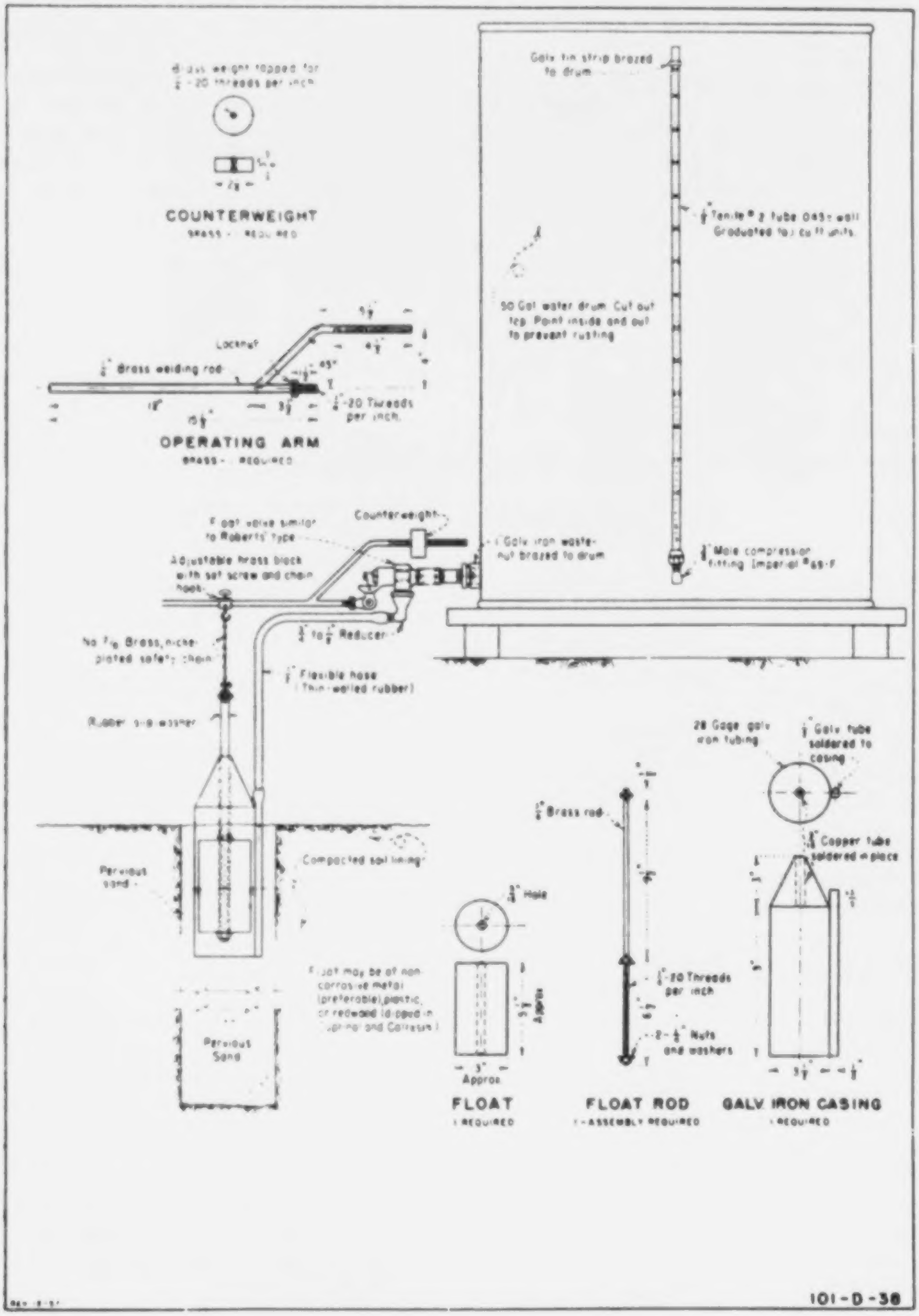

Brass weight tapped for
1/4 -20 threads per inch
COUNTERWEIGHT
BRASS - 1 REQUIRED
Locknut
1/4" Brass welding rod
1/4"-20 Threads
per inch
45°
OPERATING ARM
BRASS - 1 REQUIRED
Galv tin strip brazed
to drum
1/2" Tenite ® 2 tube .045" wall
Graduated to cu ft units
50 Gal water drum. Cut out
top. Paint inside and out
to prevent rusting
Counterweight
Float valve similar
to Roberts' type
Galv iron waste-
nut brazed to drum
1/2" Male compression
fitting Imperial ® 69-F
Adjustable brass block
with set screw and chain
hook
3/4" to 1/2" Reducer
No. 7/0 Brass, nickel-
plated safety chain
1/2" Flexible hose
(Thin-walled rubber)
Rubber slip washer
Pervious
sand
Compacted soil lining
Pervious
Sand
Float may be of non-
corrosive metal
(preferable), plastic,
or redwood (dipped in
paraffin and Cartesian)
3/4" Hole
3"
Approx
FLOAT
1 REQUIRED
1/4" Brass rod
1/4"-20 Threads
per inch
2 - 1/4" Nuts
and washers
FLOAT ROD
1 - ASSEMBLY REQUIRED
28 Gage galv
iron tubing
1/2" Galv tube
soldered to
casing
1/4" Copper tube
soldered in place
GALV IRON CASING
1 REQUIRED
101-D-38

Figure 15. - Well permeameter for measurement of soil permeability.

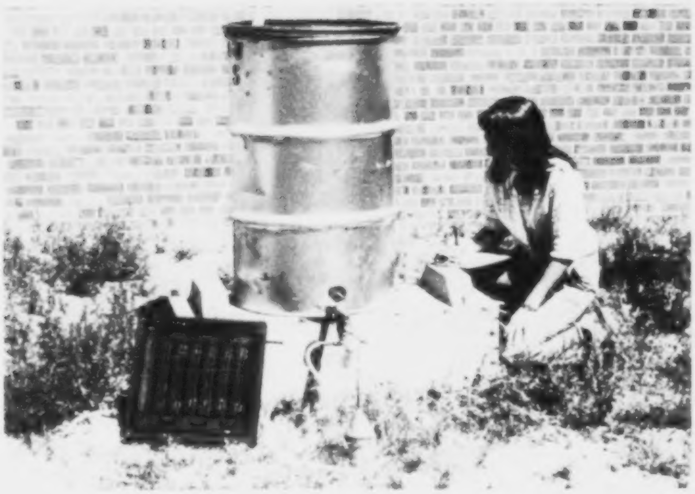


Figure 16. - A well permeameter installed for use in determining soil permeability.

Ponding Tests

The ponding method is considered the most accurate and dependable method available for determining seepage. Measurement of evaporation is necessary when ponding a lined reach in which losses are low and evaporation may be rapid, particularly if a comparison on seepage rates before and after lining is to be made. Where seepage losses are high, the evaporation of losses are often neglected because of being such a small percentage of the total loss.

Inflow-Outflow Methods

The inflow-outflow method utilizes measurements at the upstream and downstream ends of the reach being studied and is no more accurate than these measurements. Calibrated weirs or Parshall fumes can be used for measuring flows. Where permanent installations such as these are not available or not located at convenient points, temporary weirs or gaging stations can be installed. Current meters are used at gaging stations to measure the velocity from which the rate of flow is derived. Flow measurements by the inflow-outflow method are not sufficiently accurate for the close determination of seepage losses in short reaches of canal. Therefore, the longer the reach the more accurate and dependable the results will be.

Seepage Meter

Several meters have been developed for use in existing canal systems to determine seepage losses, and most are quite similar in design and functional usefulness. That commonly used by the Bureau and shown in figure 17 is a modified version of the constant-head permeameter developed for use under water. It consists of a watertight seepage cup connected with a plastic tube to a flexible waterbag. Water flows from the bag into the cup where it seeps through the 2 square feet of canal subgrade area isolated by the cup. By keeping the waterbag submerged, the heads within the areas inside and outside of the cup are equal. The seepage rate may be computed from the weight of water lost in a known period of time in the area under the meter. The seepage meter is not considered an accurate means of measuring seepage loss. Its main value lies in determining approximate locations of relatively high seepage losses. If tests are made at close intervals throughout a reach, a better indication of the average loss rate can be determined.

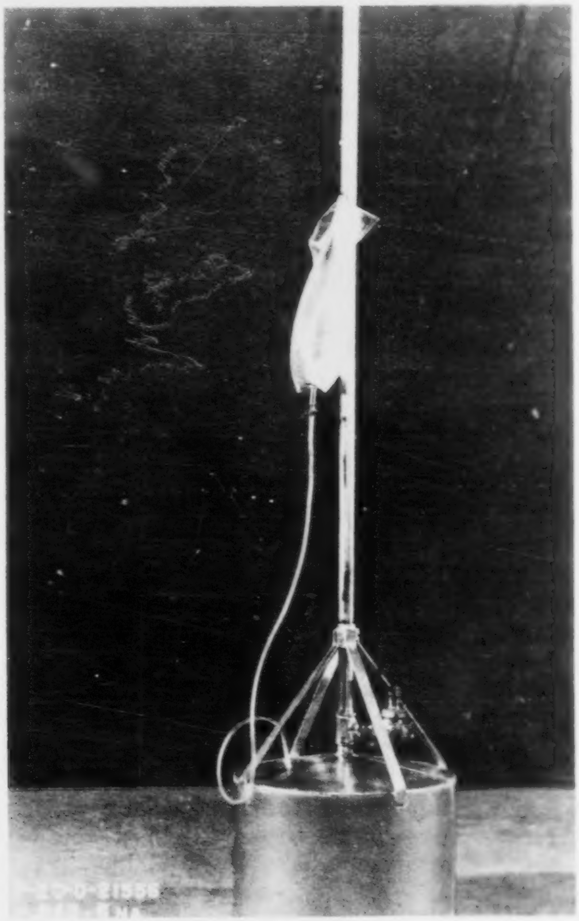


Figure 17. - Seepage meter that can be used to measure seepage losses from operating unlined canal.

REFERENCES

[1] Bureau of Reclamation Practices for Design and Construction of Concrete Lined Canals, ASCE Conference Paper, 1975

[2] Field Tests of Crack and Joint Sealing Compounds in Concrete Canal Linings - Interim Report, Report No. ChE-67, Bureau of Reclamation, Denver, Colorado, June 1966

[3] Sealing Systems for Construction Joints in Concrete Canal Lining - San Luis Canal - Central Valley Project, Report No. ChE-60, Bureau of Reclamation, Denver, Colorado, June 1966

[4] Tests on Preformed Polysulfide Strips for Canal Contraction Joints - Wahluke Branch, Columbia Basin Project, Report No. REC-OCE-69-7, Bureau of Reclamation, Denver, Colorado, December 1969

[5] Evalaution of Contraction Joint Sealing Systems - San Luis Drain - Central Valley Project, Report No. REC-OCE-70-35, Bureau of Reclamation, Denver, Colorado, July 1970

[6] Coal Tar Extended Polysulfide Canal Sealant, Report No. REC-OCE-69-1 Bureau of Reclamation, Denver, Colorado, June 1969

[7] Extension Tests on PVC Contraction Joint Forming Waterstops, Report No. ChE-103, Bureau of Reclamation, Denver, Colorado, September 1969

[8] PCV Contraction Joint Forming Waterstops - An Evaluation of Cores From the Concrete Lining in the Tehama-Colusa Canal - Central Valley Project, Report No. REC-ERC-72-30, Bureau of Reclamation, Denver, Colorado, September 1972

[9] Earth Manual, 2nd ed., Bureau of Reclamation, Denver, Colorado, 1974

[10] Performance of Plastic Canal Linings, Report No. REC-ERC-84-1, Bureau of Reclamation, Denver, Colorado, January 1984

[11] Wilkinson, R. W., "Plastic Lining on the Riverton Irrigation Project," paper presented at the ASCE Irrigation and Drainage Conference, Flagstaff, Arizona, July 1984

[12] Giround, J. P., and C. AH-Line, "Design of Earth Concrete Cover for Geomembranes"; in the Processdings of the International Conference on Geomembranes, Denver, Colorado," Vol. II, pp 487-492, June 1984

[13] Martin, Joseph P., and Robert M. Koerner, "Geotechnical Design Considerations for Geomembrane Lined Slopes: Slope Stability, Geotextiles, and Geomembranes," Vol. 2, pp 299-321, 1985

[14] Performance of Granular Soil Covers on Canals, Report No. REC-ERC-81-7, Bureau of Reclamation, Denver, Colorado, August 1981

[15] "Special Membrane Canal Lining, Farwell Main and Lower Main Canals," prepared by the Bureau of Reclamation's Nebraska-Kansas Projects Office in cooperation with Loup Basin Reclamation District, November 1982

Water Systems Operation and Maintenance Workshop - 1990

DESIGN, INSTALLATION, AND MAINTENANCE OF PIPE SYSTEMS

Session 8-1

Lowell Pimley and Gary Fischer

Design of Pipe Systems

With 66 percent of our food crops now grown west of the Mississippi, irrigation puts a great demand on the west's dwindling water supply. Irrigation accounts for more than 80 percent of the nation's daily water use. Historically, water has determined more than any other factor where America's cities have taken anchor, where America's food has been grown, and how America as a nation has been developed. We can continue to take water for granted and continue to underfinance and mismanage our water systems, or we can roll up our sleeves, clear our minds, and begin rebuilding our water systems. Meanwhile, better water management is going to be demanded. To meet this demand, projects are forced to use pipe to transport this water. The question is now asked by planners: "Have you considered a pipeline for a distribution system?" As a result of new policies and attitudes of the Bureau of Reclamation, there will be a great increase in the number of pipeline systems constructed by the Bureau in the future. A number of projects have now been constructed by the Bureau to supply water for municipal and industrial (M&I) purposes, and these use pipe systems extensively because of the value of the water involved or the need to prevent contamination.

Types of Pipe Systems

Regardless of the type of project (irrigation or M&I), there are basically three kinds of pipe systems:

1. Low-head, open-stand system
2. Low-head, closed system
3. High-head sprinkler system

Low-head, Open-stand System

In 1946 the Bureau designed and constructed a pipeline water distribution system for Coachella Valley County Water District. The Coachella Valley is in Southern California near the Salton Sea. This was an ideal place to put a pipe system because the crops are of the high-value type, there is a 12-month growing season, and water was getting more valuable.

The system we designed is what the farmers call an "underground ditch system." We normally call it a "low-head, open-stand system." Figure 1 helps illustrate the similarity between an open lateral ditch system and this system. If you notice on this schematic diagram there are a series of open stands called "baffle stands" or "pipe stands." These serve the same purpose as a check-drop structure in a ditch system. The water flows into the stand upstream of the baffle set in the middle of the stand, then the water flows over the top of the baffle, down the other side, and out through the downstream pipe. On the end of the lateral there is an overflow wasteway structure. If more water is turned into the head of the lateral than is being used by the deliveries, the water will pour out of the overflow and will be otherwise used or wasted. In this system the hydraulic gradient for design capacity as shown by the dashed line, more or less parallels the ground surface. It usually does not exceed 20 feet above the centerline of the pipe. This is so low-head unreinforced concrete pipe with mortar or rubber gasket joints can be used. Pipe sections joined together by mortar joints become one long rigid line. Hydraulic control of this type of system is quite a problem. A minimum of three gates (the main turnout, the sublateral turnout, and the farmer's delivery valve) have to be adjusted to the flow desired. Changes in delivery pattern require even more extensive gate adjustments. Moreover, as pipelines have no surface storage, changes in discharge are reflected almost immediately throughout the system. Gate adjustments must be made almost simultaneously if operational waste is to be kept within reasonable limits.

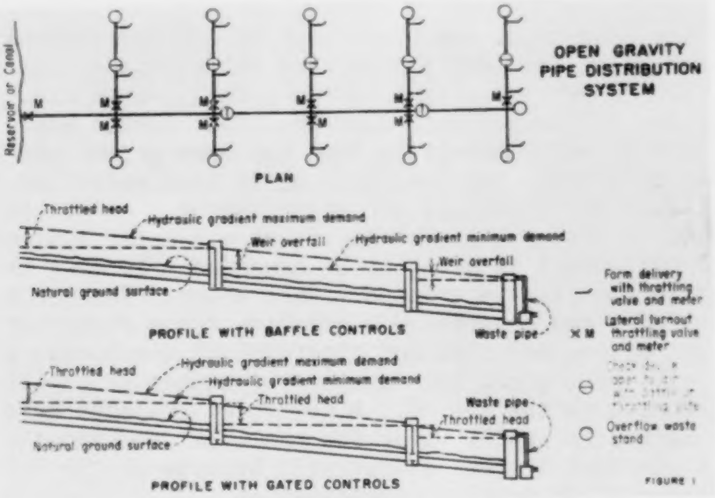

OPEN GRAVITY PIPE DISTRIBUTION SYSTEM
Reservoir or Canal
M
PLAN
Throttled head
Hydraulic gradient maximum demand
Weir overfall
Hydraulic gradient minimum demand
Weir overfall
Natural ground surface
PROFILE WITH BAFFLE CONTROLS
Waste pipe
Throttled head
Hydraulic gradient maximum demand
Hydraulic gradient minimum demand
Throttled head
Waste pipe
Throttled head
Natural ground surface
PROFILE WITH GATED CONTROLS
Farm delivery with throttling valve and meter
× M Lateral turnout throttling valve and meter
Overflow waste stand
FIGURE 1

Figure 1. - Low-head, open-stand pipe distribution system.

In the late forties, we designed similar systems for the Friant-Kern Canal distribution systems in the Central Valley of California. These included Southern San Joaquin Municipal Utility District and Ivanhoe, Lindmore, and Exeter Irrigation Districts. These systems were designed basically the same as the Coachella system, except the stands were made higher and spaced farther apart. The conditions here are not the ideal conditions for mortar-jointed pipe which were encountered in the Coachella Valley. The soil here is a heavy clay, sometimes expansive. The water, when turned into the system in the early spring after the winter shutdown, is relatively cold. The cold water causes the pipe sections to contract and thus pull apart at the joints. Deep subsidence has caused the ground surface to subside as much as 15 feet in some areas. While the maintenance costs for these systems are rather high, they are still within the budgeted amount, and the systems are considered satisfactory.

A similar system (incorporating rubber-gasketed-jointed pipe) can be used on municipal and industrial projects when the ground slope approximates the slope of the pipeline hydraulic gradient. Such a system was used on the Canadian River Project in the Texas panhandle where a 120-mile gravity-flow segment transporting water from the Amarillo regulating reservoir to the Lubbock regulating reservoir was required. A system was designed with valves controlling flow out of the Amarillo regulating reservoir and passing through a series of control structures (pipe stands and pipe checks) to maintain the pipe full of water when the control valves are closed.

This design scheme proved very economical, saving some $2.5 million (at 1963 price levels), but introduced some operational problems due to surging and overtopping under certain conditions of changes in flow. These problems were overcome by modifying some of the control structures and by establishing fairly rigid criteria for modification of flow rates, and the system has been satisfactory after these were accomplished.

This long distance transmission pipeline varies from an irrigation system in that there are very few delivery points along the line, whereas an irrigation system would have many delivery points. In general, however, the control problems are similar to those described above for irrigation systems. Figure 2 shows the profile of the central system of the Canadian River Project, and figures 3 and 4 show the types of control structures used in this part of the Canadian River Project.

Low to Intermediate-Head, Closed System

In the early 1950's, because of the increase in competition, the cost of rubber-gasketed-joint-type pipe lowered. This caused the Bureau to investigate, study, and reevaluate pipe systems. This resulted in the development of a system which eliminated the open stand and discontinued use of low-head unreinforced pipe and mortar-jointed pipe. Reinforced concrete pressure pipe with rubber-gasketed joints was used exclusively in the construction of Delano-Earlimart Irrigation District distribution system, a part of the Friant-Kern Canal distribution system. This system has performed satisfactorily for many years. It has required very little upkeep and is operated with a relatively small crew.

As you will notice in the line diagram of the plan and profile of figure 5, all open stands have been eliminated. There is no operational waste due to operation of a full-pressure system. The pipe near the downstream end must be able to withstand a greater head as the system is completely closed. This approaches a municipal-type system, except that a greater drop in pressure is allowed as usage increases. The control of flow is completely at the delivery points. They may be opened or closed as necessary, similar to you opening and closing the water faucet in your home. As you can see, the maximum gradient occurs at minimum demand. With no demand, the maximum gradient is the static elevation at the water source. The minimum gradient approaches a line parallel to the ground. (In municipal distribution systems, a higher minimum gradient would need to be provided.)

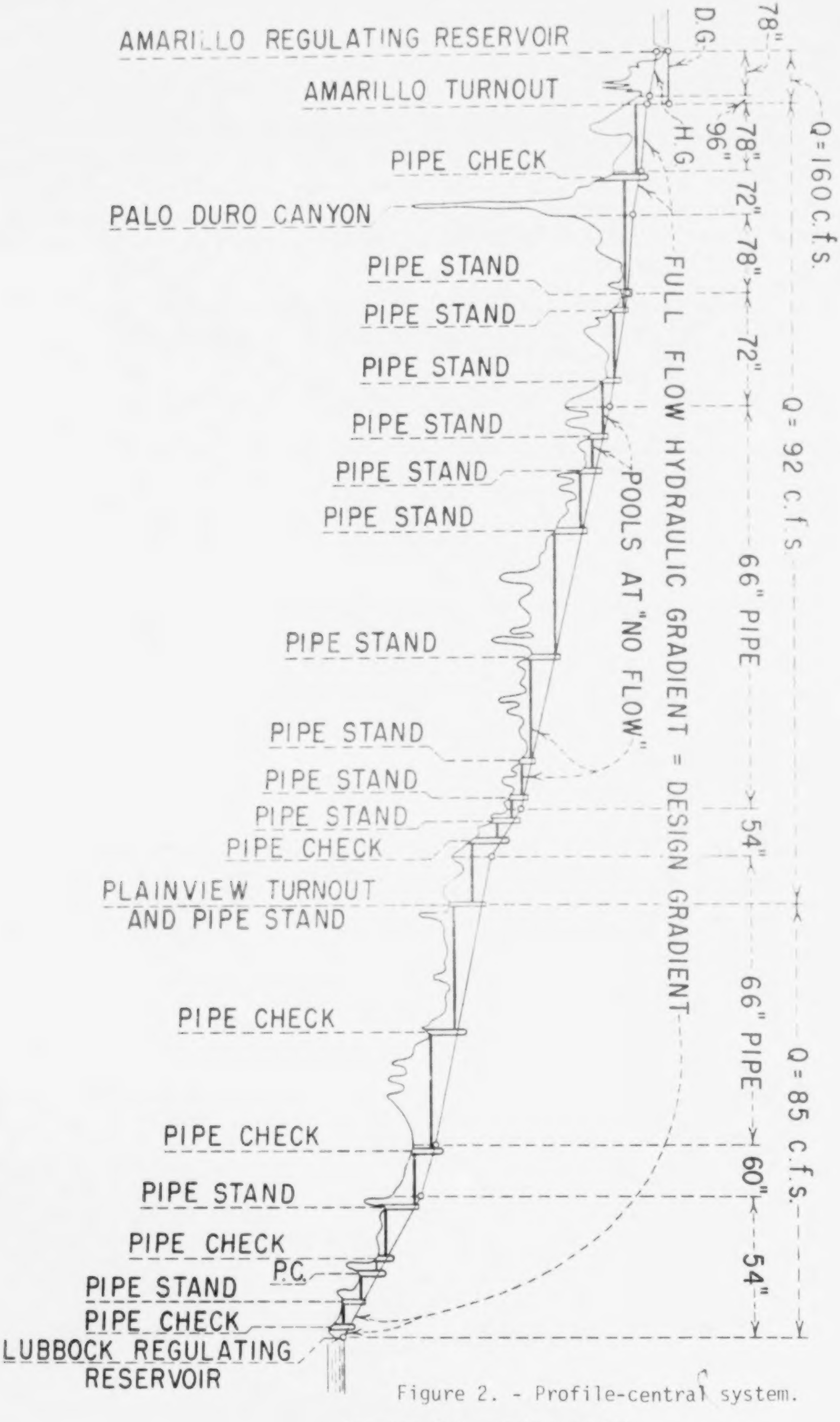

AMARILLO REGULATING RESERVOIR
AMARILLO TURNOUT
PIPE CHECK
PALO DURO CANYON
PIPE STAND
PIPE STAND
PIPE STAND
PIPE STAND
PIPE STAND
PIPE STAND
PIPE STAND
PIPE STAND
PIPE STAND
PIPE STAND
PIPE CHECK
PLAINVIEW TURNOUT AND PIPE STAND
PIPE CHECK
PIPE CHECK
PIPE STAND
PIPE CHECK
P.C.
PIPE STAND
PIPE CHECK
LUBBOCK REGULATING RESERVOIR
D.G.
H.G.
FULL FLOW HYDRAULIC GRADIENT = DESIGN GRADIENT
POOLS AT "NO FLOW"
78"
96"
78"
72"
78"
72"
66" PIPE
54"
66" PIPE
60"
54"
Q=160 c.f.s.
Q= 92 c.f.s.
Q= 85 c.f.s.

Figure 2. - Profile-central system.

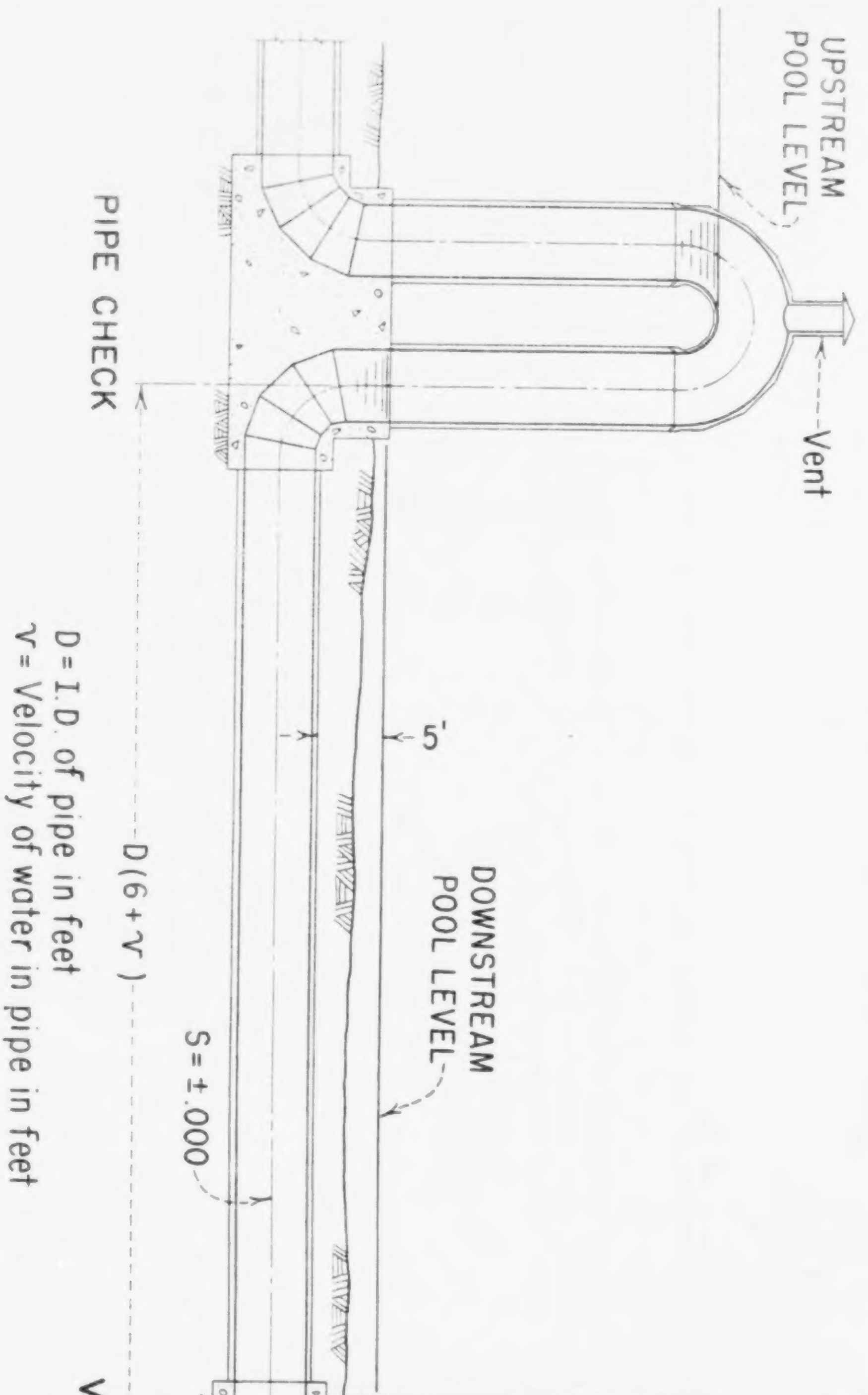

UPSTREAM POOL LEVEL
Vent
PIPE CHECK
5'
DOWNSTREAM POOL LEVEL
D(6+v)
D = I.D. of pipe in feet
v = Velocity of water in pipe in feet
S = ±.000
VENT
S = -.087

Figure 3. - Pipe check and vent.

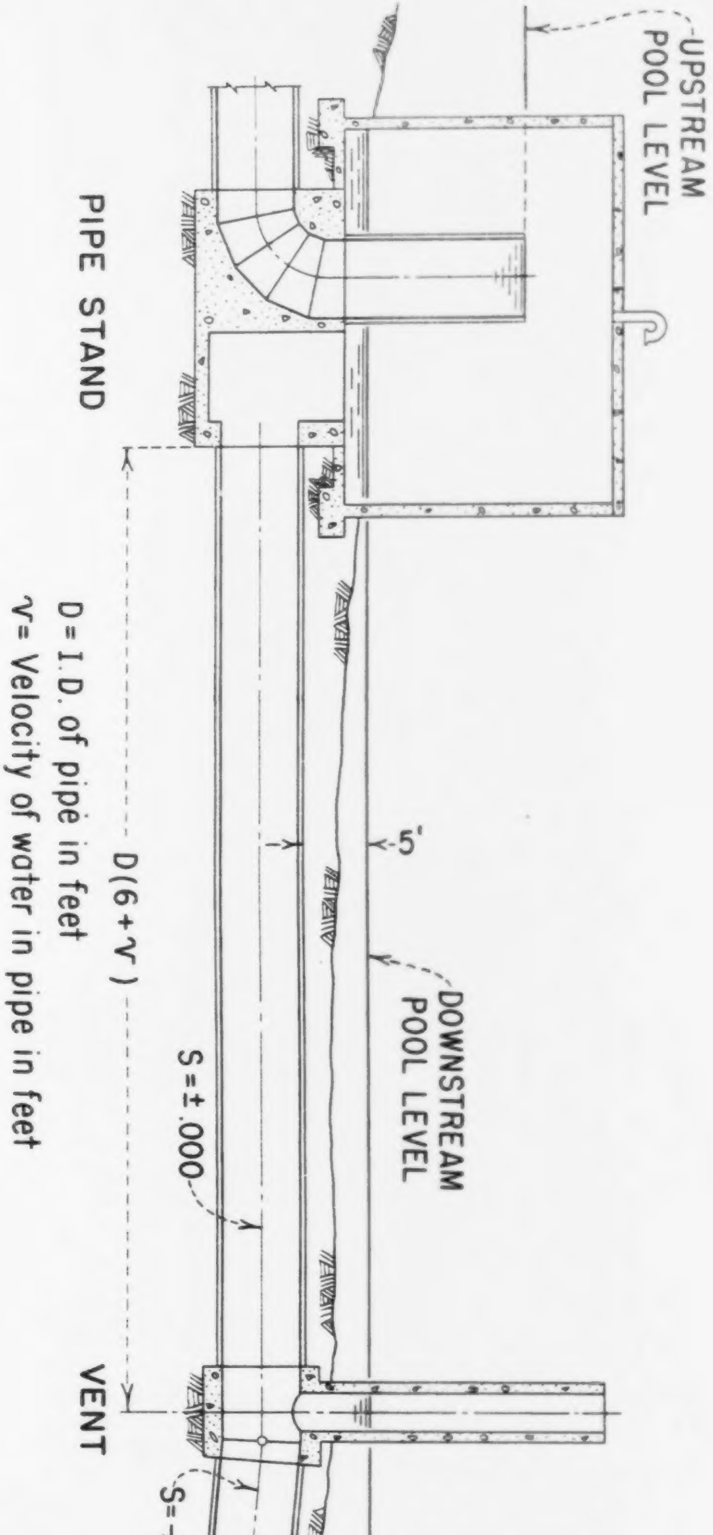

UPSTREAM POOL LEVEL
PIPE STAND
D(6+V)
D= I.D. of pipe in feet
V= Velocity of water in pipe in feet
5'
DOWNSTREAM POOL LEVEL
S=±.000
VENT
S=-.087

Figure 4. - Pipe stand and vent.

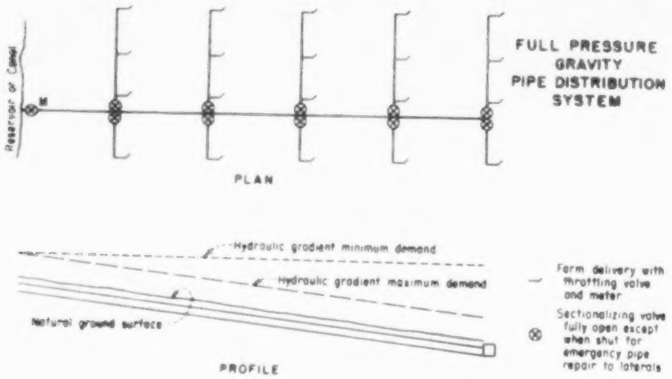

Reservoir or Canal
M
FULL PRESSURE GRAVITY PIPE DISTRIBUTION SYSTEM
PLAN
Hydraulic gradient minimum demand
Hydraulic gradient maximum demand
Natural ground surface
PROFILE
Farm delivery with throttling valve and meter
Sectionalizing valve fully open except when shut for emergency pipe repair to laterals

Figure 5. - Low-head, closed pipe distribution system.

This type of system was designed and constructed for part of the Westlands Water District. This district has an area of about three-quarters of a million acres. Approximately one-half million acres are on the downhill side below the canal, or as we say, the "gravity side" of the canal. The other quarter-million acres are above the canal or "pump side." We used a relift-type system to get water to the higher ground. Figure 6 schematically shows a system with a canalside plant and two relift plants. Each lateral was analyzed separately to get a satisfactory layout. The number of lifts for the various laterals varies from one to five in this district. The Bureau has designed and constructed approximately 1,035 miles of pipelines for the Westlands Water District.

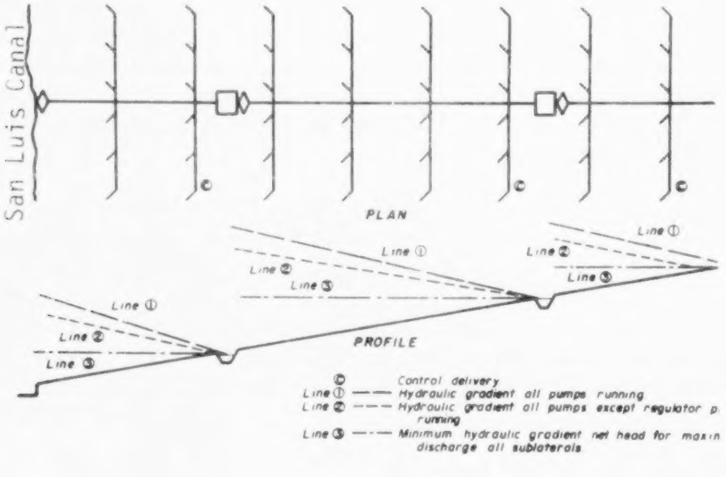

San Luis Canal
C
C
C
PLAN
Line ①
Line ②
Line ③
Line ①
Line ②
Line ③
Line ①
Line ②
Line ③
PROFILE
C Control delivery
Line ① Hydraulic gradient all pumps running
Line ② Hydraulic gradient all pumps except regulator p running
Line ③ Minimum hydraulic gradient net head for maxin discharge all sublaterals

Figure 6. - Three-lift reservoir system.

We gave the farmers a minimum of 5 feet of head at the delivery point. The point of delivery was usually located on the high point of their land or at least somewhere on the high side. From this point the users constructed their own system for distributing the water on their land. Generally, they did one of three things. They let the water dump on the ground into an open ditch and distributed it in open laterals. They connected a pipe to the delivery and distributed their water by pipe or a combination of pipe and open ditch. Some of the farmers connected booster pumps to the delivery, which enabled them to use a sprinkler system.

This type of closed system can also be used in municipal and industrial projects where automatic controls can allow cities or industries to withdraw their deliveries as needed, with the amounts withdrawn automatically through operation of control valves or pumping plants. For example, the Southern System of the Canadian River Project (south of Lubbock, Texas) incorporates a closed system with pressure reducing stations. The pressure reducing stations feed water into regulating tanks or terminal storage tanks and operate to automatically supply the volumes of water needed by the cities, while regulating the maximum rates of flow and maintaining the pipeline pressure within acceptable limits. Figure 7 illustrates the profile of this system, showing the design and hydraulic gradients and the static water levels provided by the pressure reducing stations. Figure 8 shows the pressure reducing station at regulating tank No. 6 just south of Lubbock.

High-head Pressure-demand System

More and more farmers are using the sprinkler type of system. Many in the Central Valley of California are using this method. Every year large numbers are converting to sprinkler systems. In your drives around the countryside you have probably seen similar type systems in operation. Sprinkler systems are more efficient, require less manpower, and conserve water. In this day of depleting water supplies and of increasing difficulty in getting competent help, these are very important. Since sprinkler systems require fairly high pressure for operation, pipelines supplying water for sprinklers must be designed for high minimum heads, just as municipal distribution systems must have available substantial pressure for each customer. This need for "pressure on demand" dictates the use of elevated storage tanks and other features which will provide the pressure necessary to operate sprinklers.

The Columbia Basin Project in the northwest was originally designed and constructed as an open-ditch-type system, with the farmers also distributing by open ditches. About 60 percent of the farmers in this basin have installed pumps and use sprinkler systems. Landowners in one large undeveloped area of this basin are now considering the installation of a pipe distribution system, giving them sprinkler pressure so they can directly hook up to the system. This will eliminate the necessity of farmers installing and operating their own pump.

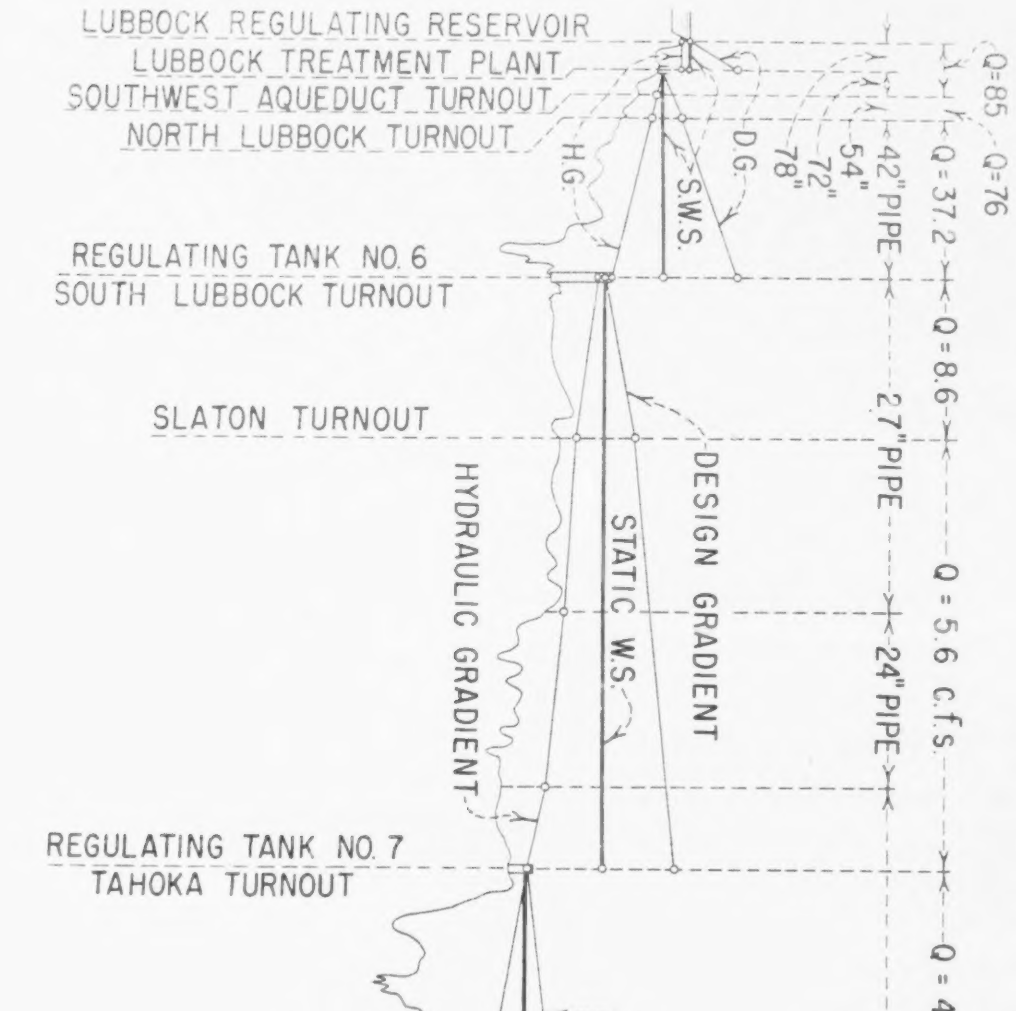

LUBBOCK REGULATING RESERVOIR
LUBBOCK TREATMENT PLANT
SOUTHWEST AQUEDUCT TURNOUT
NORTH LUBBOCK TURNOUT
H.G.
S.W.S.
D.G.
78"
72"
54"
42" PIPE
Q=37.2
Q=85
Q=76
REGULATING TANK NO. 6
SOUTH LUBBOCK TURNOUT
Q=8.6
SLATON TURNOUT
27" PIPE
DESIGN GRADIENT
STATIC W.S.
HYDRAULIC GRADIENT
Q = 5.6 c.f.s.
24" PIPE
REGULATING TANK NO. 7
TAHOKA TURNOUT
Q = 4.7 c.f.s.
DESIGN GRADIENT
O'DONNELL TURNOUT
21" PIPE
STATIC W.S.
HYDRAULIC GRADIENT
Q = 4.2 c.f.s.
18"
LAMESA TERMINAL

Figure 7. - Profile - Southern system.

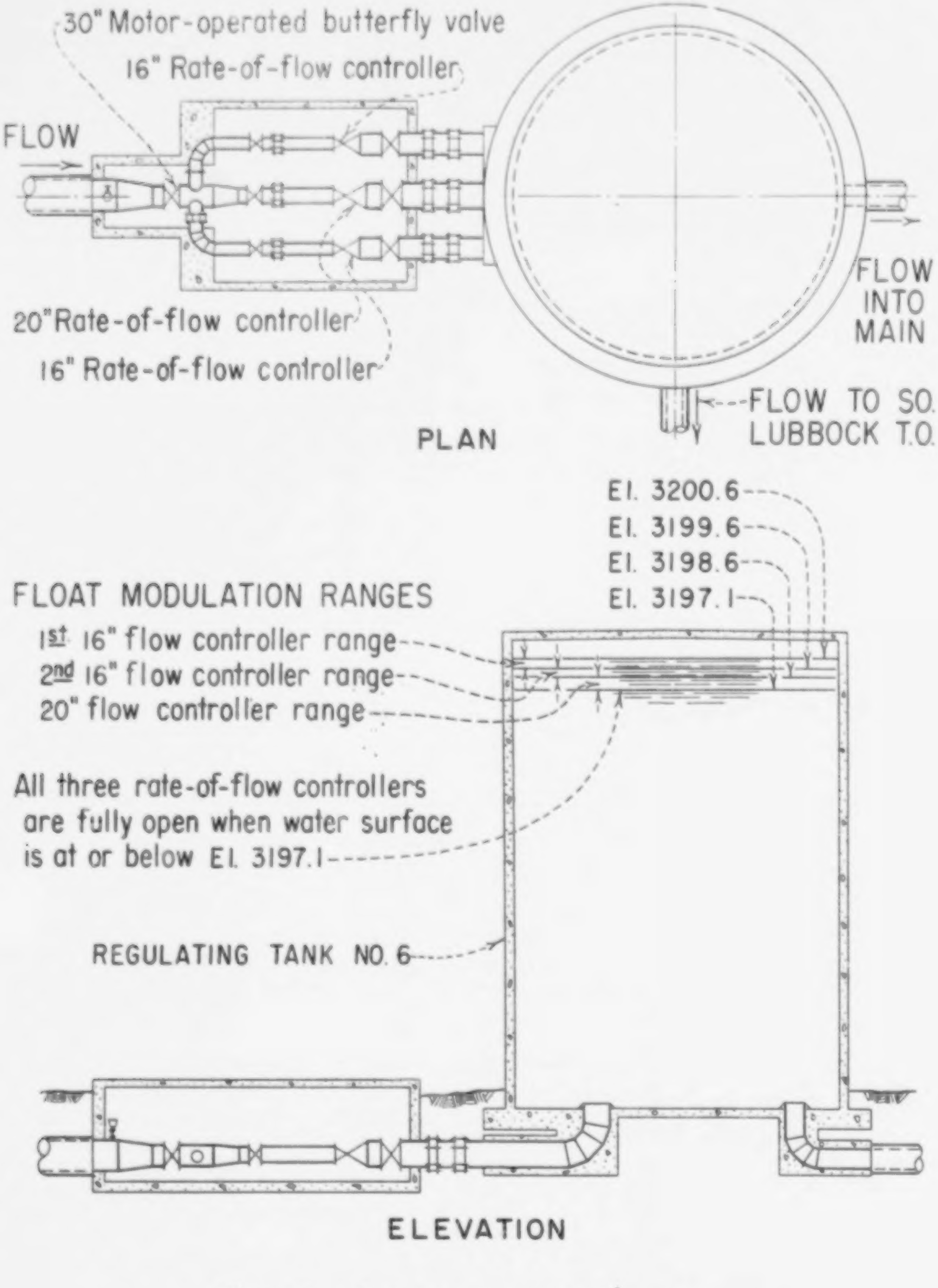

30" Motor-operated butterfly valve
16" Rate-of-flow controller
FLOW
20" Rate-of-flow controller
16" Rate-of-flow controller
FLOW INTO MAIN
FLOW TO SO. LUBBOCK T.O.
PLAN
El. 3200.6
El. 3199.6
El. 3198.6
El. 3197.1
FLOAT MODULATION RANGES
1st 16" flow controller range
2nd 16" flow controller range
20" flow controller range
All three rate-of-flow controllers are fully open when water surface is at or below El. 3197.1
REGULATING TANK NO. 6
ELEVATION

Figure 8. - Pressure reducing station at regulating tank No. 6.

The trend toward the use of high-cost, automated sprinkler irrigation systems is causing reduction in labor requirements and more efficient use of water which is becoming more valuable every day. Figure 9 illustrates how a pipe system affects the duty of water for a hypothetical cropping pattern.

A major cause for inefficient use of water, particularly with surface irrigation systems, is the high labor cost coupled with inexpensive water. Rather, the farmer often adjusts water application time to labor patterns dictated by general farm operations. The 12-hour irrigation set practiced by many farmers, particularly during nighttime hours, often results in excessive percolation and runoff losses where soils are coarse textured and intake rates are high. Furthermore, excessive use of water may lead to drainage and salinity problems that are hard to alleviate.

The Bureau has installed several high-pressure systems of this type serving over 30,000 acres. One of these systems, Spokane Valley Project, Washington, was completed in 1967. The source of water in this case is not a canal or river, but underground water. Figure 10 shows a typical well field. You can see four motors and concrete platforms. These platforms support deep well pumps underneath. The trapdoors you see cover a pit containing controls, meters, valves, etc. The object in the upper right is part of one leg of an elevated tank. Figure 11 gives a complete picture of the tank. The tank controls pressure surges in the pipe which occur when the pumps go on and off. Also, there is a float control in the tank for each pump. This control is set to start the pump when the water in the tank gets low and to stop the pump when the water gets to a higher elevation. These control points are staggered; that is, each pump starts and stops at a different water level in the tank. This keeps them from all starting or stopping at the same time.

Figure 12 is a picture of a typical sprinkler system on the Spokane Valley Project. This farmer is irrigating grass pasture. You can see the light aluminum, movable pipes on top of the ground. These sprinklers are directly connected to the district system at the point of delivery at the border of the land parcel.

We are currently working on a sprinkler-type system for the NIIP (Navajo Indian Irrigation Project) which is located in northwestern New Mexico. The project area consists of 110,630 acres which is divided into 11 blocks of land approximately 10,000 acres each. The first five blocks have been constructed and are in an operational status. One lateral system in Block 6 is under construction now and the remainder of Block 6 is being designed.

The average pressure at the sprinkler nozzles is 55 pounds per square inch. The farm unit turnout pressures provide for a minimum of 63 pounds per square inch at any point along the field mainline. The terrain, size, and shape of each of the farm units are considered when selecting type of sprinkler system.

TYPE CONVEYANCE FARM APPLICATION	PRESSURE PIPELINE SPRINKLER	PRESSURE PIPELINE ROW FLOODING	CANAL ROW FLOODING
Farm efficiency	•70 %	65 %	65%
Farm hdgle diversion	3.57 AF	3.85 AF	3.85 AF
Conveyance efficiency trunklines and laterals	97 %	97 %	70 %
Annual irrigation requirement from dam or stream	3.68 AF	3.98 AF	5.50 AF
Overall efficiency	68 %	63 %	46 %

Assume average crop consumptive use = 2.5 AF (acre-foot) per acre

Figure 9. - Comparison of overall irrigation efficiency of a pipe system to open lateral systems.

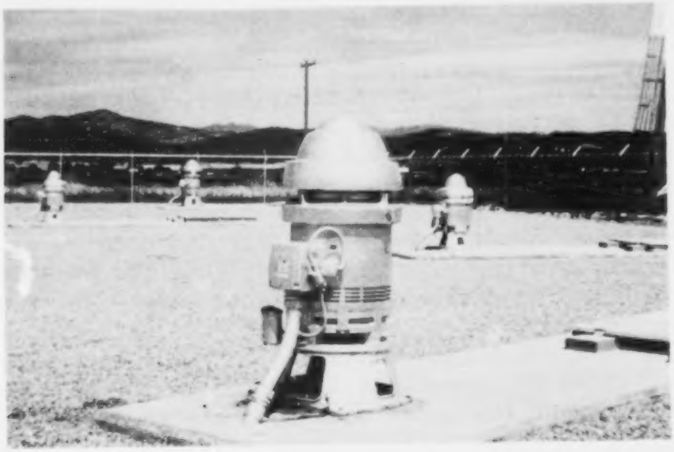


Figure 10. - Well field, Spokane Valley Project, Washington.

Figure 11. - Elevated water tank, Spokane Valley Project, Washington.

Figure 12 - Typical farmer's irrigation system, Spokane Valley Project, Washington.

Each delivery installation on the NIIP has a sectionalizing valve, meter, and pressure-reducing valve. The water is measured and the flow and pressure are automatically controlled to each farm unit. When a power outage occurs, each delivery will automatically close to prevent draining the pipe distribution system.

Pump units in the multiunit pumping plants cycle automatically with pressure fluctuations in the system. The pumps in most of these pumping plants are regulated by elevated tanks which have off-on switches or floats for each of the pumps. On a few small lateral systems, each delivery has a single pump dedicated to it. These pumps are located at one pumping plant site. A jockey pump and hydropneumatic tank maintain a constant pressure in the pipe system.

Variable speed pumps and motors have been used on some of the lateral systems. Programable controllers signal speed controllers which regulate the operation of the variable speed pumps and motors. This concept of using variable speed units is fairly new to the Bureau. As we gain more experience with variable speed pumps and motors, we hope to be able to utilize them more often.

Type of Pipe Used in Irrigation Systems

Once the type of pipe system desired has been determined, the type of pipe to be used is the next important question to be resolved. Selection of the type of pipe or pipes to be used in the system is made during the designing stage by the designing engineer and your district. To increase the competition among contractors who bid on the specifications, ordinarily a combination of types of pipe are selected instead of only one pipe type. The main types of pipe within the head range being used on Bureau projects are:

Type of Pipe	Diameters (inches)	Heads (feet)
Reinforced concrete pressure pipe	12 through 108	25 through 150
Embedded cylinder prestressed concrete pipe	48 through 108	25 through 700
Noncylinder prestressed concrete pipe	60 through 108	25 through 700
Lined cylinder prestressed concrete pipe	16 through 48	25 through 500
Pretensioned concrete cylinder pipe	10 through 54	25 through 700
Steel line pipe - mortar-lined, coal-tar enamel coated and wrapped; or mortar-lined and mortar coated	4 through 108	All heads
Reinforced plastic mortar pipe	8 through 54	25 through 500
Ductile-iron pipe	4 through 54	25 through 800
Polyvinyl chloride pressure pipe	4 through 27	25 through 700

The Bureau has found by experience that the pipe joint is one of the most important features in a pipe distribution system. A good pipe joint must be flexible enough to permit longitudinal movement due to temperature changes in the water and to wetting and drying of the pipe, as well as vertical adjustments due to settlement that sometimes occurs due to the application of irrigation water to adjacent lands.

Low-head pipe initially used on Bureau pipe distribution systems utilized tongue-and-groove precast concrete pipe with a mortar-banded, rigid-type joint. Pipelines laid with this joint have a low first cost and, generally, a higher maintenance cost.

The rubber-gasketed pipe joints were developed by the various pipe manufacturers to meet requirements established by the Bureau. Figure 13 shows outlines of the most commonly used types of joints. The concrete or steel element containing the gasket is constructed and shaped in such a way that the gasket is contained in a groove and is not required to support the weight of the pipe. When the pipe is centered, the round gasket is compressed into the groove, forming a watertight seal. See figures 14 through 16 for drawings and pictures of some of these pipes and their joints.

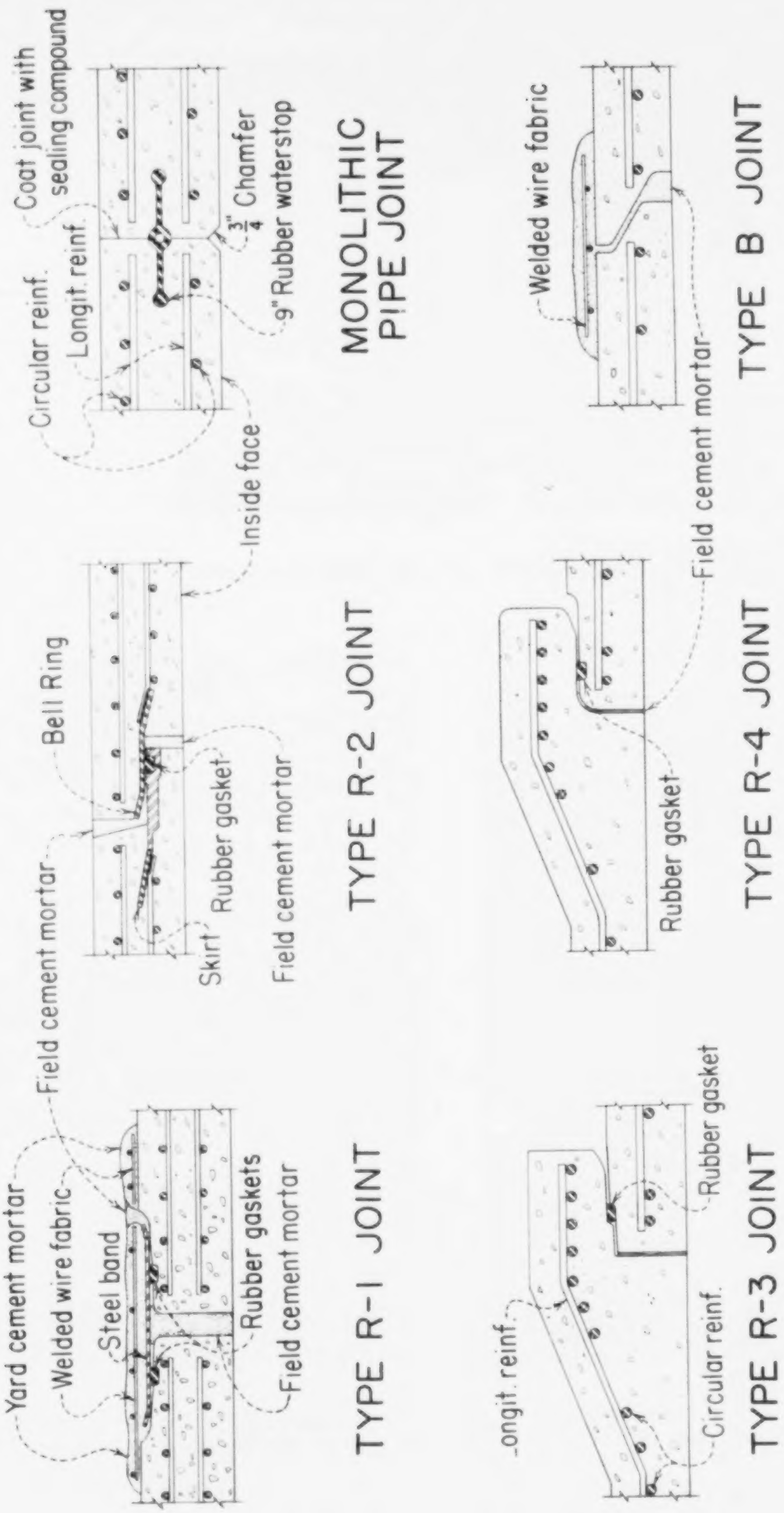

Yard cement mortar
Welded wire fabric
Steel band
Rubber gaskets
Field cement mortar
Field cement mortar
TYPE R-I JOINT
Field cement mortar
Skirt
Rubber gasket
Field cement mortar
Bell Ring
Inside face
TYPE R-2 JOINT
Circular reinf.
Longit. reinf.
Coat joint with sealing compound
3/4" Chamfer
9" Rubber waterstop
MONOLITHIC PIPE JOINT
ongit. reinf.
Circular reinf.
Rubber gasket
TYPE R-3 JOINT
Rubber gasket
Field cement mortar
TYPE R-4 JOINT
Welded wire fabric
Field cement mortar
TYPE B JOINT

Figure 13. - Concrete pipe joints.

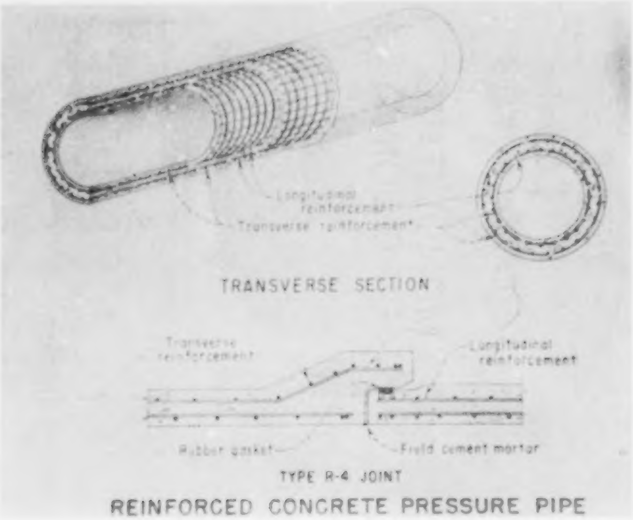

Longitudinal reinforcement
Transverse reinforcement
TRANSVERSE SECTION
Transverse reinforcement
Longitudinal reinforcement
Rubber gasket
Field cement mortar
TYPE R-4 JOINT
REINFORCED CONCRETE PRESSURE PIPE

Figure 14. - Reinforced concrete pressure pipe.

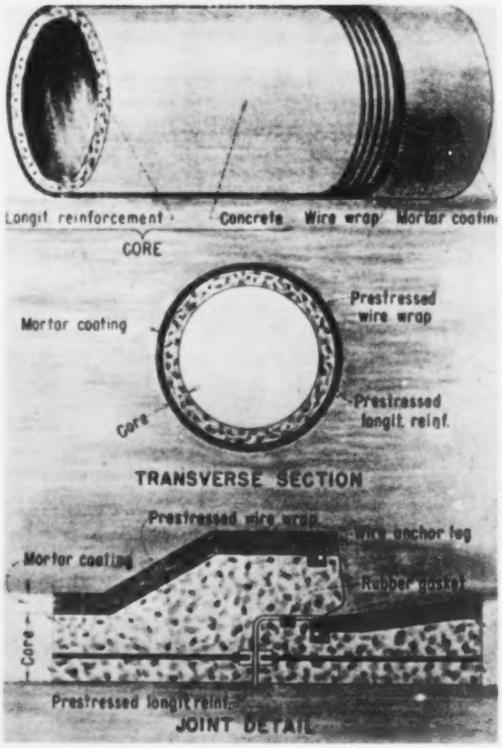

Longit reinforcement
Concrete
Wire wrap
Mortar coatin
CORE
Mortar coating
Prestressed wire wrap
Prestressed longit. reinf.
Core
TRANSVERSE SECTION
Prestressed wire wrap
Wire anchor lug
Mortar coating
Rubber gasket
Core
Prestressed longit reinf.
JOINT DETAIL

Figure 15. - Pretensioned concrete pipe.

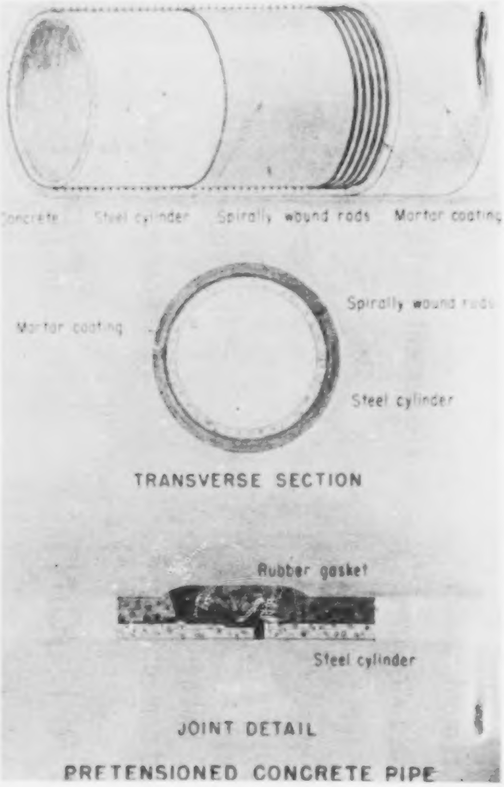

oncrete
Steel cylinder
Spirally wound rods
Mortar coatin
Spirally wound rod
Mortar coating
Steel cylinder
TRANSVERSE SECTION
Rubber gasket
Steel cylinder
JOINT DETAIL
PRETENSIONED CONCRETE PIPE

Figure 16. - Noncylinder prestressed concrete pipe.

The Bureau divides the pipe types into two groups, flexible and rigid.

Type	Diameter (inches)
Flexible Group	
Pretensioned concrete cylinder pipe	21 through 54
Steel line pipe	12 through 108
Reinforced plastic mortar pipe	12 through 108
Ductile-iron pipe	24 through 54
Polyvinyl chloride pressure pipe	4 through 27
Rigid Group	
Reinforced concrete pressure pipe	12 through 108
Embedded cylinder prestressed concrete pipe	60 through 108
Noncylinder prestressed concrete pipe	60 through 108
Lined cylinder prestressed concrete pipe	16 through 48
Pretensioned concrete cylinder pipe	10 through 18
Steel line pipe	4 through 10
Reinforced plastic mortar pipe	8 and 10
Ductile-iron pipe	4 through 20

The ultimate performance of any pipe installation depends not only on the quality of the pipe itself, but also on the correct execution of sound trench, bedding, and backfill requirements which constitute an integral part of the pipe design.

Rigid pipe is designed for a bedding angle of 90°. Achieving a firm bedding is essential to the satisfactory performance of the pipe. Compaction of backfill around the bottom of the pipe to a depth of 0.37 of the outside diameter is required by the specifications to insure that adequate bedding is obtained and to assure that the design load distribution occurs under the pipe. It is essential that adequate compaction of the backfill is performed under the pipe haunches in order to insure that the 90° bedding angle is obtained. As an example, the external load-carrying ability of a rigid pipe with line bedding at the bottom is roughly half that of a pipe having proper bedding around the bottom 90°.

For flexible pipe, still greater restraint requirements are required from the surrounding soil. For a flexible pipe to achieve its design strength and maintain its circularity, the soil must provide side support for the pipe walls in resisting the vertical earth loads. Because of this, a greater amount of compacted backfill is required around flexible pipe than rigid pipe. Flexible pipe requires a high degree of compaction around its sidewalls to a minimum height of 0.7 of the outside diameter to develop passive soil resistance and to prevent excessive vertical deflection under full trench loads. Polyvinyl chloride pipe requires additional compaction of backfill to a depth of 6 inches above the pipe. This greater depth of compacted backfill is necessary because of the characteristics of the polyvinyl chloride pipe in the smaller size range (4 inches to 10 inches).

A few Bureau projects have been constructed with pipes bedded in soil-cement or mortar. In these instances, the pipe trench is excavated to a semicircular section providing an annular space of 2 to 3 inches between the pipe and the trench walls. This space is filled with a soil-cement or mortar slurry designed to provide a compressive strength of approximately 1,000 lbs/in^2 after curing (see figure 17). The support provided by the hardened slurry and the undisturbed foundation material eliminates the need for compaction of backfill at the haunches and provides the necessary support for the pipe walls. Pipes installed using this procedure experience very few failures which can be traced to improper bedding, and seem to bear up well under imposed loads from above.

When leaks are encountered in pipe bedded in this manner, however, the need for removal of the bedding to effectuate proper repair, and the subsequent requirement for replacement of the bedding after repair, can cause a problem in the repair process.

Corrosion is the big enemy of pipes containing ferrous materials. Requirements for controlling external corrosion on buried pipe alternatives containing ferrous elements can be determined from corrosion surveys. These surveys should determine the soil resistivities and stray currents along the pipeline alinement, as well as an assessment of corrosion history in the area. Auxiliary corrosion control requirements should be specified in accordance with the indicated corrosivity as follows:

1. Pipes Containing a Ferrous Cylinder

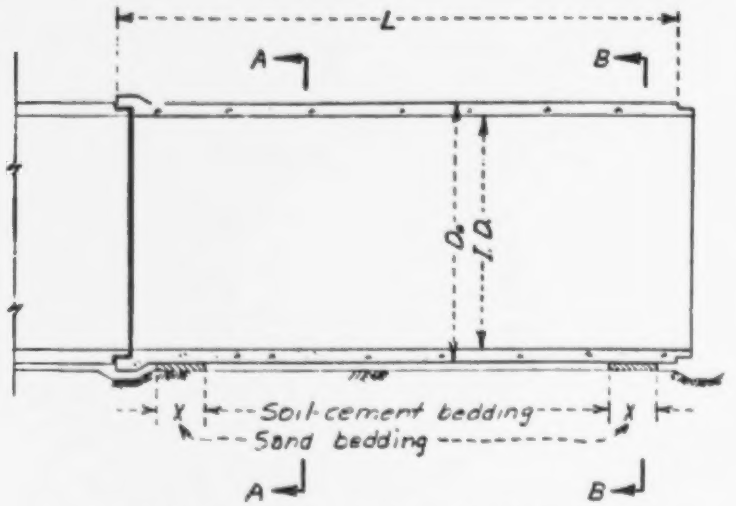
Severely corrosive. - Installation of a complete cathodic protection system is required.

Moderately corrosive. - Installation of a corrosion monitoring system and the design for a cathodic protection system are required. The corrosion monitoring system allows the determination of pipeline corrosion parameters and facilitates the application of cathodic protection, if required at some future date.

Mildly corrosive. - No auxiliary protection beyond that provided by the protective coating is required.

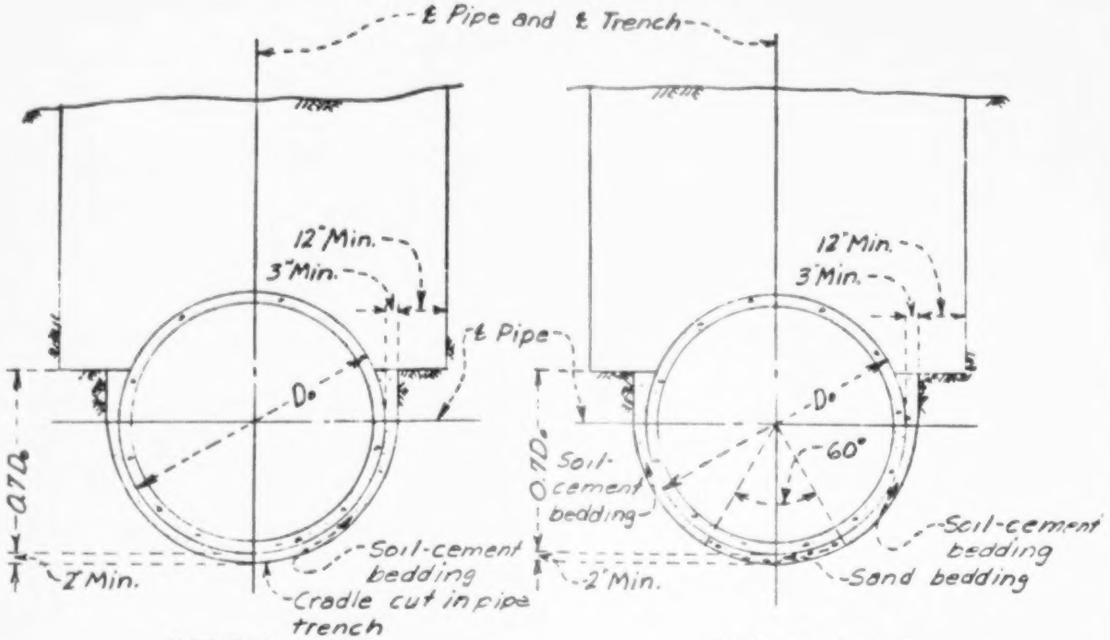
2. Reinforced Concrete Pressure Pipe

Severely corrosive. - The corrosion resistance of this pipe is attributable to embedment of all the reinforcing steel in concrete and the subsequent lack of electrical continuity across the pipe joints. However, this pipe remains susceptible to


L
A
B
Do
I.D
X
Soil-cement bedding
Sand bedding
A
B

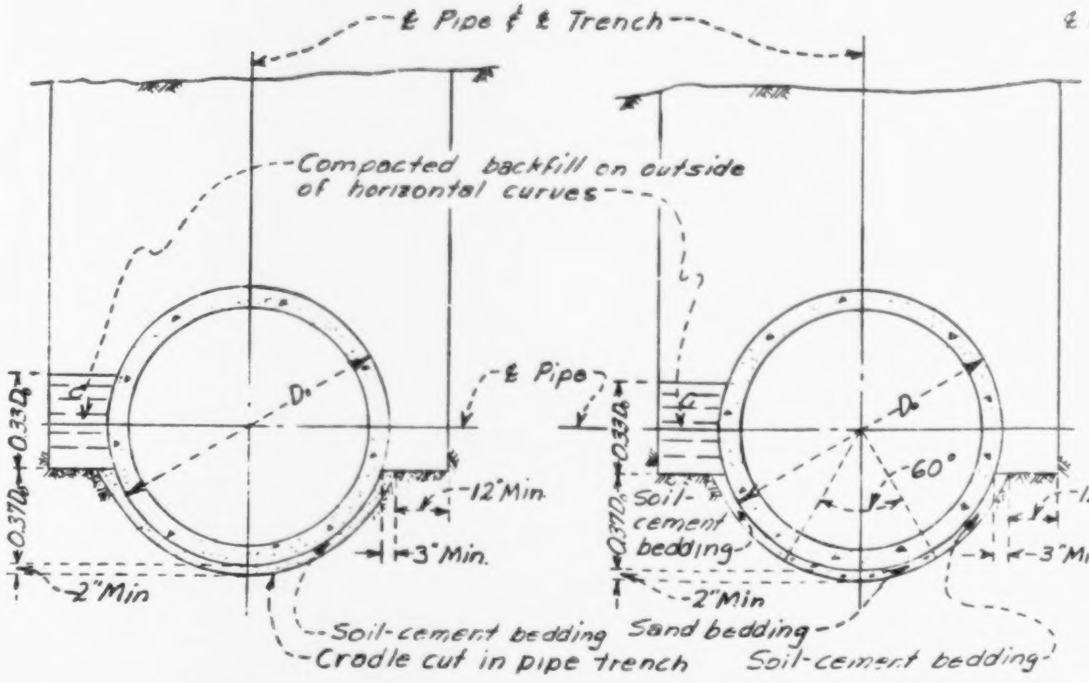
LONGITUDINAL PIPE SECTION

PIPE I.D	X
14"	.05L
16"	.05L
18"	.05L
20"	.06L
21"	.06L
24"	.07L
27"	.07L
42"	.09L
54"	.10L
72"	.12L


℄ Pipe and ℄ Trench
12" Min.
3" Min.
X
Do
0.7Do
2" Min.
Soil-cement bedding
Cradle cut in pipe trench
℄ Pipe
Soil-cement bedding
60°
X
2" Min.
Soil-cement bedding
Sand bedding

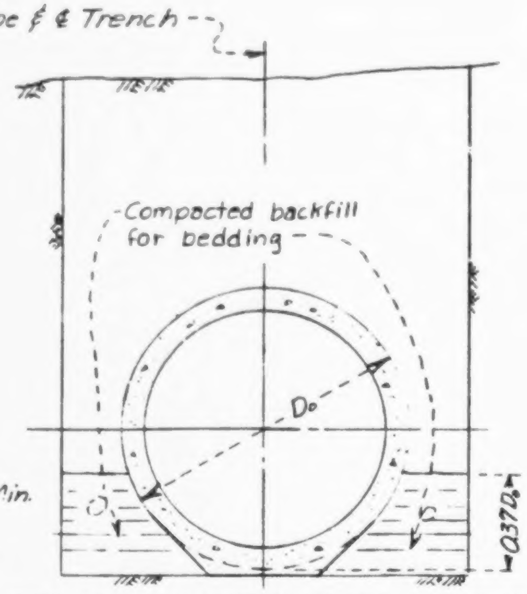
SECTION A-A SECTION B-B

PRETENSIONED CONCRETE PIPE 21" AND LARGER
STEEL PIPE 12" AND LARGER


℄ Pipe & ℄ Trench
Compacted backfill on outside of horizontal curves
℄ Pipe
Do
0.33Do
0.37Do
12" Min
3" Min.
2" Min
Soil-cement bedding
Cradle cut in pipe trench
Do
60°
0.33Do
0.37Do
Soil-cement bedding
12" Min.
3" Min
2" Min
Sand bedding
Soil-cement bedding

SECTION A-A SECTION B-B

CONCRETE PRESSURE PIPE 12" AND LARGER.
PRETENSIONED CONCRETE PIPE 12" TO 18" INCL.
NON-CYLINDER PRESTRESSED CONCRETE PIPE
ASBESTOS-CEMENT PIPE 12" AND LARGER


℄ Pipe & ℄ Trench
Compacted backfill for bedding
Do
0.37Do

PRE-BED CONCRETE PRESSURE PIPE
(For other pipe trench requirments and notes, see Dwg. 662-D-1011

NOTES

Place pipe to line and grade on sand bedding as shown in Sec. B-B. Place soil-cement bedding under remainder of pipe as shown in Sec. A-A

Soil-cement bedding shall be placed under pipe from one side and rodded or vibrated so that it flows under pipe until it appears on other side. Then mortar shall be added to both sides and rodded or vibrated until it completely fills the space between the pipe and cradle of the pipe trench. This operation shall follow as closely behind pipe setting operations, shown in Longitudinal Pipe Section and Sec. B-B, as is feasible. Where required to prevent uplift, the soil-cement bedding shall be placed in two stages as directed, allowing sufficient time for the initial set of the first stage before the remainder is placed.

For Do see Dwg 662-D-1011.

2-5-65 D KLP.	ADDED: ASBESTOS-CEMENT PIPE 12" AND LARGER.
10-7-64 D KLP.	CHANGED SAND AND MORTAR BEDDING TO SAND BEDDING, LONGITUDINAL PIPE SECTION.
7-1-64 D KLP.	MORTAR BEDDING CHANGED TO SOIL-CEMENT BEDDING ADDED COMPACTED BACKFILL ON OUTSIDE HORIZ CURVES

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CANADIAN RIVER PROJECT - TEXAS

MAIN AQUEDUCT, LUBBOCK TO LAMESA, AND SOUTHWEST AQUEDUCT

PIPE TRENCH ALTERNATE

DRAWN R.D.L. SUBMITTED F.C. Olander
TRACED RECOMMENDED H.E. Rippon
CHECKED R.S. APPROVED O.F. Rice
CHIEF DESIGNING ENGINEER

DENVER, COLORADO, MARCH 16, 1964
662-D-1012

corrosion particularly in very low resistivity soils and under stray current conditions. Should severe corrosion be experienced, no practical methods of control exists since cathodic protection is ineffective when used on reinforced concrete pressure pipe.

Appurtenances

Structures appurtenant to pipelines include air valves which are placed at each high point to exhaust air during filling and admit air during draining, blowoffs which are placed at each low point in the line to facilitate draining, control structures such as pipe stands and pipe checks which are discussed above, wasteways, surge tanks, moss screen or trashrack structures installed at the inlet to pipelines from canals or reservoirs, and pumping plants to lift water to higher elevations when necessary.

Figure 18 is a picture of a moss screen and recirculating pump structure at the head of a lateral in the Ivanhoe Irrigation District. This part of the equipment is the traveling moss screen structure. The source of water is the Friant-Kern Canal which runs adjacent to the structure. The screen is similar to an endless belt and is self-cleaning. The water in a pipe system must be fairly clean, otherwise the flow measurement meters and valves will get fouled up. On the downhill side of the canal, there are usually some parcels of land near the canal which are too high to be delivered to by gravity. Small, low-head pump systems are used to get water to these lands. The vertical stand is a recirculating stand for the low-head pumps set on the deck of the structure. If more water is being pumped than is being delivered to the farmers, the excess water goes into the overflow on the stand and back into the pump sump.

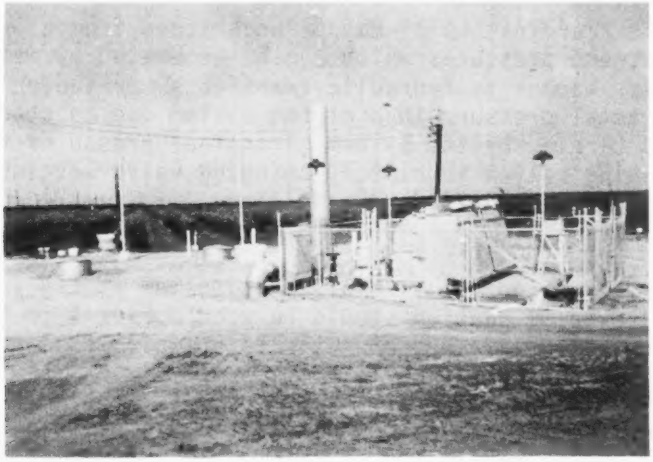


Figure 18 - Moss screen and pumping plant, Ivanhoe Irrigation District, California.

In a pipe system, the pipe represents about 70 to 80 percent of the cost. The appurtenances such as moss screens, low-head pumping plants, and so on, make up the remainder.

Installation

For any of the various pressure pipes, it is most important that care be taken while the pipe is being installed. This includes the bedding and the backfilling afterward. Many of the problems which occur later in operating a system are caused either by careless or incompetent work during the initial installation.

The joints, with their rubber gaskets, must be carefully installed, and the pipe must have a uniform bearing for its full length. The bedding material must be carefully placed in accordance with the requirements of the specifications. Special attention must be given when pipe enters or leaves a structure or is connected to a valve in order that differential settlement will not take place, causing breaks which are extremely difficult to repair.

Maintenance

Anyone operating a new full-pressure system must staff and train capable people to protect the system and the water users' interests.

After construction is completed, the designers stress to the operators a

"CARDINAL RULE"

FILL SLOWLY - OPERATE VALVES SLOWLY - SHUT DOWN SLOWLY

The importance of this "Cardinal Rule" may be understood from a brief explanation of the extreme pressures which can be generated by water hammer. The definition of water hammer is hydraulic transients or sudden upsurges or downsurges of the total pressure in a piping system due to changes in velocity of the fluid carried by the system. Transient pressures build up due to opening or closing valves to rapidly changing valve settings (accidental or planned), starting or stopping pumps, and unstable pump turbine characteristics.

The most critical period during the life of the pipeline is the initial filling and testing. Extreme caution should be used during this time period. During the filling period, a safe procedure is to not allow the velocity to exceed 0.25 ft/s (Bureau/E&R Center criteria).

Opening and closing valves too rapidly can literally blow pipe out of the ground and cause extensive damage. The same result can occur if air is trapped in pipelines by excessively rapid filling and failure to vent all air from the system. Close observation of air valves during draining and filling and adherence to Designers' Operating Criteria in the operation of valves, pumps, and control facilities is absolutely necessary to prevent damage. Avoiding sudden changes in velocity will generally avoid serious water hammer surges.

In a warm, nonfreezing climate, preventive maintenance on the pipes themselves consists of keeping them full of water at all times during the nonirrigation season. In a cold, freezing climate, all the pipes must be drained or buried below the frost line and kept empty during the cold months. This is, of course, not possible on M&I systems which operate year round or on pipeline systems located in a flood plain which may cause the pipe to float out of the ground. For these systems, freeze protection is included in design and some flow should always be maintained to combat freezing. If conditions have changed since the system was designed and flows cannot be maintained, the system must be protected from freezing, especially the air release valve and PRV (pressure reducing) valve. One district in the Pacific Northwest Region has installed an antifreeze-type solution in the air release valve to prevent it from freezing similar to those used for mobile homes and RV campers.

During the nonirrigation season, large diameter pipe should have protective barriers placed at the inlet and outlet ends to prevent people and vehicles from entering the pipelines. Small pipelines should also be protected to prevent rodents from entering the lines during the nonirrigation season. Siphons under roads should also be protected, especially if they are left full of water. Protective barriers should be placed at each end to prevent small children or animals from entering the structures.

Periodic inspections of large diameter pipe interiors and exteriors and necessary repairs to the coatings must be made when needed. Culverts under canals should be inspected to determine if there are any misalignment cracks or if seepage is occurring.

Low-head, irrigation-type gate valves can cause a considerable amount of maintenance. Broken gate leaves, arches, and stems are not uncommon; and replacement parts are not always from the same pattern or the valve cannot be adjusted to stop leakage. The trend upon replacement is toward a higher class valve with removable seats.

Several irrigation districts in the Pacific Northwest Region are now using butterfly valves instead of gate valves on pump installations. Operating personnel report that butterfly valves are easier to throttle, are providing better service, and are cheaper to purchase.

Pipeline Leaks - Causes and Repairs

Principal causes of leakage in precast concrete pipes (reinforced or unreinforced) are broken backs or bellies, joint failures, broken collars, splits, temperature cracks, and settlement at structures. Broken backs or bellies (circumferential cracks) can result from improper handling during installation, excessive loading from backfill or traffic, settlement or heave, and other causes. These are the most commonly experienced leaks to occur in precast concrete pipe.

Joint failures can result from rolled or pinched rubber gaskets, or from cracked bell or spigots. These can occur as a result of improper installation, settlement or heave, or excessive loading under some conditions. Joint failures commonly are more difficult to repair than circumferential cracks.

Splits, which are longitudinal cracks, are sometimes caused by the pipe expanding in length either from warming or absorption of moisture. Splits occur more often in cast-in-place pipes than in precast pipes. Precast pipe can be split by improper operation (opening or closing valves too fast, malfunction of pump controls, etc.) which results in exceeding design pressures.

Temperature cracks are most prevalent in unreinforced concrete pipe but can cause trouble in reinforced pipes as well. These cracks take place when abnormally cold water is introduced into the pipes. The pipes contract and literally pull themselves apart. The cracks are usually transverse and will vary in width from hairline to one-fourth inch. The possibility exists that temperature cracks are aggravated by laying the pipe in extremely hot weather without sufficient protection. It may also be true that temperature cracks are mistaken for broken backs or vice verse.

Uncontrolled corrosion can cause serious leakage of precast concrete pipe containing a ferrous cylinder. If undetected, corrosion can completely destroy the cylinder and reinforcing rods leaving only the mortar lining to carry the water under pressure. Such corrosion can require extensive repair up to and including replacement of entire sections of pipe.

In low-head systems, it is sometimes difficult to locate actual leaks or even to know a leak exists. Minor leaks may be more accurately described as seeps with moisture never reaching the ground surface. Under such conditions, the leakage would be considered as a pipeline loss much the same as leakage through cracks or joints in concrete-lined open ditches.

Repair would be attempted only if losses became excessive or damage to foundation resulted. In some cases, particularly where pipelines traverse steep slopes, water from leakage may travel underground through the pipe trench and reach the surface considerable distances away from the actual leak. A small hand soil auger sometimes is useful in locating the origin of the water.

Among various ways to stop pipeline leaks, the introduction of material into a pipeline in operation to act as a sealant for small cracks is quick and easy. One of the earliest sealants used was horse manure. Today, sawdust is sometimes used. Sawdust flour and coarser ground sawdust are mixed with water and pumped into the line. Naturally, these procedures cannot be used on M&I systems and in any case are only temporary repairs. If the water is removed from the lines, chances are they will leak again when refilled.

The introduction of NH_3 (anhydrous ammonia) appears to have a great deal of promise. The success of the system depends on the precipitation of calcium carbonate when ammonia is introduced. Unfortunately, the action does not occur with all waters. The Bureau initiated an investigation of ways and means of introducing other materials into water along with ammonia to promote precipitation. A report of this method has been published in Release No. 67 of the Bureau's Operation and Maintenance Bulletin.

A method commonly used to repair transverse broken backs or bellies in irrigation and reinforced concrete pipe is to chip or chisel the crack to form a V-notch one-half to three-fourths inch wide and deep. Leakage is stopped by calking with lead wool. The notch is thoroughly cleaned by wire brush or possibly sandblasting. An epoxy-base compound or mortar is troweled into the notch. The exterior of the notch is troweled to form a relatively uniform surface. Thin sheet rubber is then wrapped around the crack. The sheet rubber generally has a Shore durometer rating of from 30 to 40 compared to approximately 55 for the rubber gasket material. The sheet rubber is covered with a thin metal band and cinched up using narrow steel bands. After cleanup and drying of the area, the foundation is compacted and an unreinforced concrete encasement is placed around the repair. For added protection, a concrete cradle is placed under the pipe approximately 18 inches each side of the encasement.

Commercially available pipe repair clamps can often be used, without the need for chipping and calking, where the circumferential crack occurs in pipe with smooth concrete surface. These clamps can often be installed without taking the line out of service, and utilize bonded waffle-surfaced rubber which will effectively seal the leak permanently. Placement of a mortar covering over the metal material of the clamp is recommended.

Commercial clamps become extremely expensive for pipe diameters of 30 inches and larger, and some success with repairing circumferential cracks in large diameter pipes has been experienced using locally fabricated clamps. Normally, 3/16- or 1/4-inch steel plate is rolled to the required diameter in two or three sections, depending on the size of pipe involved. These are installed over pads of "camel-back" rubber similar to that used for vulcanizing tires.

Rods or straps encircling the pipe and clamp are used for tightening. Again, a mortar coating is applied after completion of the repair. These clamps have the distinct advantage of maintaining flexibility while avoiding the excessive weight imposed by a concrete collar or encasement. As with any type repair, replacement of bedding to the original specifications requirements is important to prevent future problems.

Joint failures can sometimes be repaired by calking the interior of the pipe joint with a mastic material which maintains its flexibility indefinitely. (This type repair is useful at low line pressure where easy access to the pipe interior is available.) Various types of field fabricated clamps to reinforce and/or encase broken bells or spigots have been attempted, but no great success has been experienced with repairing these types of pipeline leaks. Encasement of the joint in a concrete collar is sometimes necessary, although not considered desirable.

Rubber-gasketed joint leaks caused by pinched gaskets can also sometimes be stopped under pressure by caulking with lead wool from the outside. If the joint moves after calking, there is a good possibility that eventually the joint will leak again. "Freezing" the joint by placing concrete encasement after calking often may cause shearing next to the pour and should be avoided if possible. However, some districts have been successful with this method.

Repair of pipes containing a steel cylinder may sometimes be made by welding, either using a partial or complete "butt-strap" closure sections or commercially available repair collar (see figure 19). In some cases, replacement of complete sections may be the most economical and most desirable method of repair. Figure 20 illustrates a replacement closure section with manhole to allow for mortar coating the interior of the closure section. This same type of closure can sometimes be adapted for installation in other types of pipe when a concrete section must be replaced.

Some districts have arrived at the conclusion that one cannot effect a permanent cure for leaks in a mortar-jointed concrete pipeline having to operate under low water temperatures. These districts are replacing their poor lines with rubber-gasketed pipe of concrete or PVC (plastic pipe) whichever is the most competitive on a year-to-year basis.

Maintenance of Appurtenances

All air valves should be examined regularly to assure that they function properly. Air valves which do not operate regularly can stick shut and fail to operate when needed. If the hydraulic pressure at an air valve location is low, the air valve may not seal properly and continued leakage will damage the seating surfaces. During drainage and refilling, air valves should be checked for proper operation and to be sure that all air is exhausted after the system is filled. If gate valves are used below the air relief valve, you may want to remove the wheel from the valve stem so unauthorized closures will not occur.

Blowoff valves should be operated periodically to assure the valves will function when needed. In cold climates, blowoff valves which are equipped with discharge piping should be opened with a blind flange installed at the end of the discharge pipe) so that freezing of water in the blowoff pipe will not crack the valve housing.

Backfill adjacent to air valves and blowoff structures and insulation on the underside of concrete tops must be adequately maintained to prevent freezing. Whenever a repair is made which requires removal of insulation from an air valve, the insulation should be properly replaced upon completion of the repair.

Maintenance and necessary repair of other concrete structures should be performed as needed. Leakage through tank or structure walls should be stopped and repairs made to prevent spalling and deterioration of the concrete. Metalwork should be painted regularly and grounding systems as well as other protective features should be maintained in satisfactory condition. If backfill material is removed from the valve enclosure, it should be replaced to provide insulation.

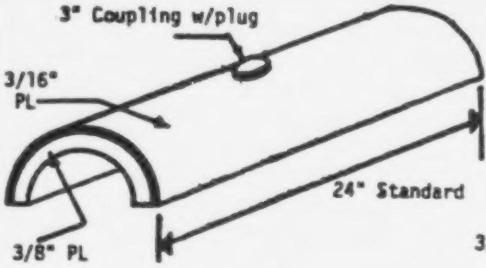
When provided, corrosion monitoring facilities should be serviced and pipe-to-soil potentials measured regularly. Test stations (including anode sites) should be measured regularly and depleted anodes replaced. Periodic review of all corrosion monitoring readings by a knowledgeable engineer is very important to assure that corrosion does not gain an insurmountable advantage and that the need for corrective or preventive measures is determined early.

GIFFORD-HILL-AMERICAN, INC.

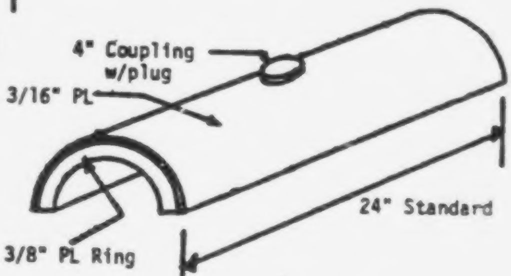
DWG. No. 12
REF. No. REPAIR
PREP. BY : DWK
DATE : 6-15-79

NON-GASKET REPAIR SADDLE
FOR P-303 AND L-301 PIPE

I. The Repair Saddle
  A. Top Piece -
    3/16" PL x 24" Long-Rounded (180°) Steel Plate
    w/3" Coupling w/Plug
    Attached to two-3/8" PL 180° Rings
    (Interior to be coated with Dimetcote E-Z)
  B. Bottom Piece -
    3/16" PL x 24" Long-Rounded (180°) Steel Plate
    w/4" Coupling w/Plug - Attached to two-3/8" PL 180° Rings
    (Interior to be coated with Dimetcote E-Z)

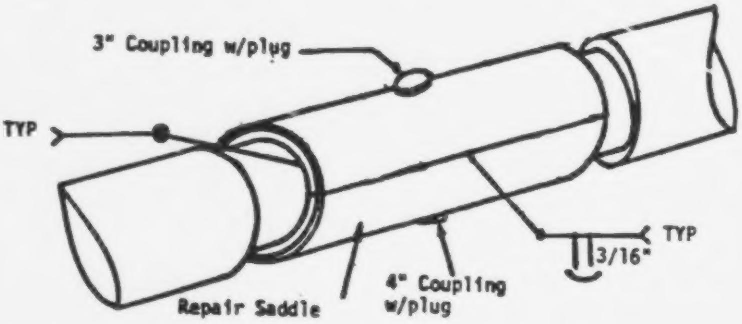

3" Coupling w/plug
3/16" PL
24" Standard
3/8" PL

A


4" Coupling w/plug
3/16" PL
24" Standard
3/8" PL Ring

B

II. The Assembled Repair Saddle


3" Coupling w/plug
TYP
TYP
3/16"
Repair Saddle
4" Coupling w/plug

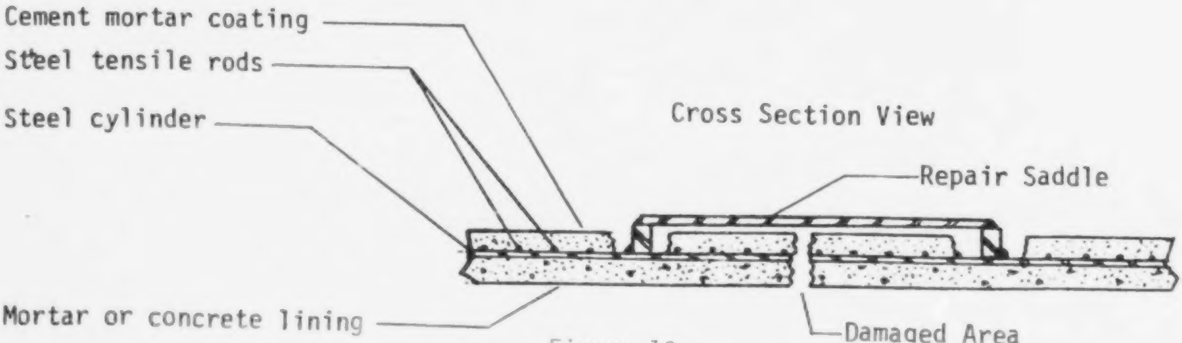

Cement mortar coating
Steel tensile rods
Steel cylinder
Cross Section View
Repair Saddle
Mortar or concrete lining
Damaged Area

Figure 19

GIFFORD-HILL-AMERICAN , INC.	DWG. No. 27
CLOSURE SECTION	REF. No. REPAIR
	PREP. BY : DWK
	DATE : 6-19-79

I. THE CLOSURE SECTION

A. SHORT PIPE w/ACCESS OPENING

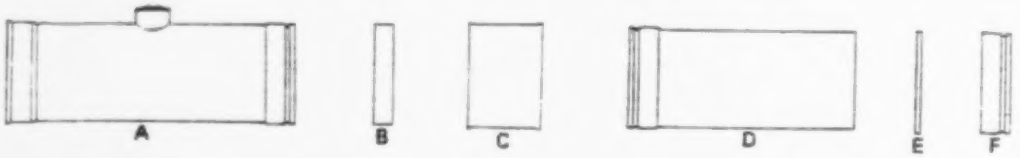
B. STEEL FILLER RING

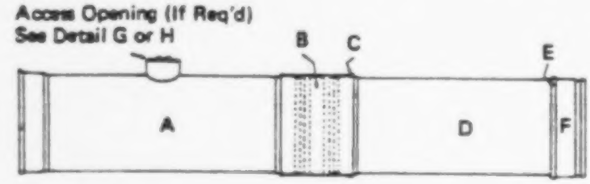
C. STEEL CYLINDER SLEEVE

D. SHORT PIPE w/PLAIN END

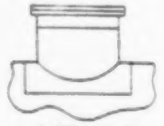
E. STEEL BACK-UP RING

F. JOINT RING



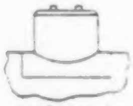


18" FLG. OUT. w/BLIND FLG



DETAIL G

6" CPLG w/PLUG



DETAIL H

II. INSTALLATION PROCEDURE

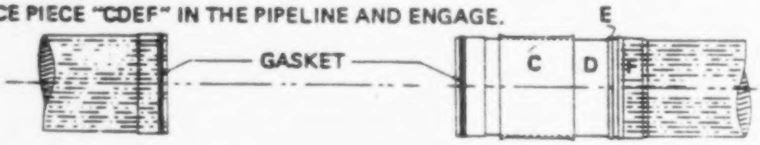
1) MEASURE CLEAR SPACE DISTANCE BETWEEN JOINTS OF EXISTING PIPE.

2) CUT PIECE "D" TO THE REQUIRED LENGTH.

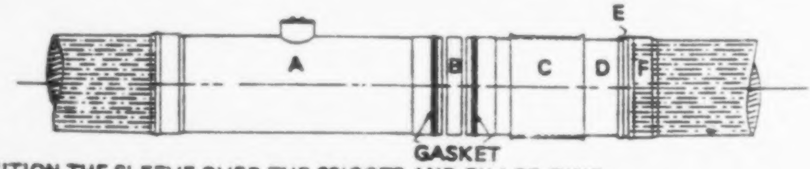
3) WELD A STEEL BACK-UP RING (PIECE "E") TO THE PLAIN END OF PIECE "D", AND WELD THE JOINT RING (PIECE "F") TO THE BACK-UP RING.

4) PLACE THE SLEEVE (PIECE "C") OVER THE SPIGOT OF PIECE "D" AND SLIP BACK TO EXPOSE THE GASKET GROOVE.

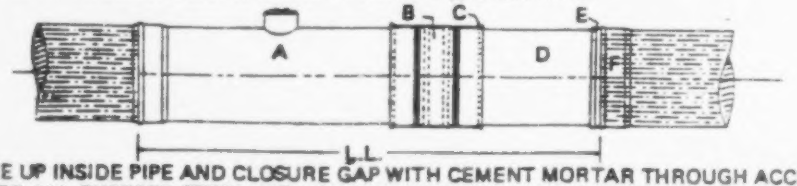
5) PLACE PIECE "CDEF" IN THE PIPELINE AND ENGAGE.



6) PLACE PIECE "A" IN THE PIPELINE AND ENGAGE. PLACE FILLER RING IN REMAINING GAP.



7) POSITION THE SLEEVE OVER THE SPIGOTS AND FILLER RING.



8) MAKE UP INSIDE PIPE AND CLOSURE GAP WITH CEMENT MORTAR THROUGH ACCESS HOLE AND COVER ALL EXPOSED STEEL WITH CEMENT MORTAR (1" MINIMUM).

Figure 20

Water Systems Operation and Maintenance Workshop - 1990

CONCRETE MIXES AND MATERIALS

Session 9-1

Jeff Hart and Bill Kepler

I. Properties of Concrete

   A. Fresh Concrete

      1. Workability
      2. Consistency
      3. Segregation
      4. Bleeding

   B. Hardened Concrete

      1. Strength
      2. Durability
      3. Volume Change

II. Ingredients of Concrete

   A. Water
   B. Cement and Pozzolan
   C. Aggregates
   D. Admixtures

III. Use of table 19, Concrete Manual (Mixes for Small Jobs)

IV. Demonstration - Effects of Dirty Aggregates on Concrete

BUREAU OF RECLAMATION
DIVISION OF RESEARCH AND LABORATORY SERVICES
CONCRETE AND STRUCTURAL BRANCH

PROPERTIES OF FRESH CONCRETE

by

J. S. Pierce*

In a previous session we defined portland cement concrete, but now let us further subdivide that into fresh and hardened concrete. I think the definition of the latter term is apparent; i.e., it is concrete in which the chemical reaction of hydration has occurred and the concrete is hardened or rigid. It will not regain its original shape after deformation. However, fresh concrete and its properties is the subject I want to concentrate on during this session. Then in the next session we will discuss the properties of hardened concrete.

To me fresh concrete is concrete which has not lost its plasticity. Most normal concrete under average ambient conditions will remain fresh for approximately 2 hours. At least I think you would agree that it could be termed fresh until initial set, and that would average around 2 hours. The British have actually written in a Standard that concrete is fresh for 2 hours after the addition of water.

Many people interchange the terms "fresh" and "plastic"; however, you have to be careful because we now produce a true plastic concrete. This plastic or polymer concrete will be discussed in some depth next week. Thus, you see why we refer to plastic concrete as fresh concrete.

Since concrete service does not begin until it has reached a hardened state, you may wonder why we are interested in the properties of fresh concrete. Realistically, concrete performance is largely dictated by its fresh properties. First, I want to define those properties of interest and then briefly describe some tests that measure those properties.

One of the most important properties is workability. Many people interchange this term with slump which actually is a measure of consistency. Workability includes consistency, but it is more complex than consistency. Workability is a nondimensional quality that can be defined as the ease with which concrete can be mixed, transported, placed, and finished with a minimal loss of homogeneity. You can deduce from this that workability is a function of the flow properties of concrete. Consequently, people often think of slump as a measurement of workability, but if you are concerned with mixing, transporting, placing, and finishing, then you need more than a single measurement test such as slump to directly measure workability. Factors that control the degree of workability include the following:

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*Chief, Concrete and Structural Branch

1. Mix design parameters such as water content, water-cement ratio, percentage of sand, and mix temperature
2. Aggregate size, shape, texture, porosity, and gradation
3. Type and dosage of admixtures
4. Cement fineness
5. Amount of mixing and time elapsed since mixing

From this list you can see that although workability is a very important property, we cannot define it quantitatively. Instead we must use qualitative terms such as high, low, etc.

When we speak of the slump test as being a measure of consistency, we are speaking of the relative mobility or ability of fresh concrete to flow under one particular force. Consistency measures the degree of wetness or fluidity of a given concrete. Thus, a high consistency could mean low workability due to segregation with a very wet mix.

Segregation, which is the lack of homogeneity, can be a function of the concrete itself or it may result from improper handling and placing techniques or a combination of both. When segregation occurs, the distribution of ingredients is no longer uniform and you cannot fully consolidate the mix nor accurately determine its properties. Segregation commonly occurs in mixes that are either too wet or too dry. With these types of mixes, the grout and coarse aggregate readily separate resulting in rock pockets and sand streaks. If you experience segregation, look for the source of the problem among the following factors which increase concrete's tendency to segregate:

1. Increases in maximum aggregate size over 1 inch
2. Increases in the proportion of the largest aggregate fraction
3. Increases in coarse aggregate density relative to fine aggregate density
4. Decreases in the amount of aggregate fines

5. Decreases in the cement content
6. Changes in aggregate shape
7. Changes in the water content

The volume of voids has a great effect on the ultimate properties of concrete. Voids occur as entrapped air, entrained air, and as the result of inadequate consolidation. Entrapped air is present to some extent in all mixes and is a function of the aggregate size and shape. Smaller maximum-size aggregates (MSA) have higher percentages of entrapped air; e.g., a 3/8-inch MSA has approximately 3 percent entrapped air, while a 6-inch MSA has only about 0.2 percent. Entrained air is intentionally introduced into the concrete with an admixture. These voids differ from entrapped air in size, shape, and distribution and provide several benefits such as improved durability and workability, reduced permeability, and water requirement. For all of these benefits there is a slight penalty in strength since the air replaces solid ingredients. The benefits far exceed the strength penalty and the Bureau requires that all concrete contain entrained air.

Bleeding occurs due to the instability of dense, solid particles dispersed in water. The solids will settle and the water will rise leaving a layer of water on the surface of the concrete. Due to the nature of its occurrence, the process is called bleeding. It is actually a form of segregation occurring after the concrete is placed. You might think that the water movement would reduce the water-cement ratio and be desirable, but it is not a uniform water loss. Actually, a water gain occurs in the top half of the concrete, thereby increasing its water-cement ratio and lowering the quality. The rising water also carries fine, inert particles of sand and cement to the surface which preclude development of good bond strengths. This layer of inert fines is called laitance and is what is removed during sandblasting. Another problem with bleeding occurs in reinforced concrete. The water migration collects under the rebar and when it evaporates, it leaves a void and reduces the bond to the concrete. If finishing operations start while bleed water remains on a slab, the layer of water gets trowelled into the concrete and creates a zone of weakness and usually results in a scaling slab. When you investigate bleeding, you should look at both the rate and total amount of bleeding. Due to the nature of concrete, bleeding occurs to some extent with all mixes. However, if you experience deleterious amounts of bleeding, the following corrective actions should reduce the rate of bleeding:

1. Decrease the water content
2. Increase the cement fineness

3. Increase the cement content, but this may actually increase the total amount of bleeding
4. Use a pozzolan
5. Use an air-entraining admixture

Some people say you can control the rate of bleeding by adjusting the sand content, but fines retained on a No. 200 sieve have practically no effect on the rate of bleeding. So with clean aggregates, sand adjustments will not help much. Dirty aggregates with a high percentage passing the No. 200 sieve do have an appreciable effect though.

Both fresh and hardened concretes experience volume changes. In the fresh state, the volume change is termed plastic shrinkage. It occurs as random, deep, straight cracks in the concrete before it hardens. When the evaporation rate exceeds the bleeding rate, plastic shrinkage results due to the rapid drying at the surface. At this point, essentially no strength has developed in the green concrete, so it cannot resist even the rather low stresses developed by drying. Evaporation is a function of the relative humidity, wind velocity, and temperatures of the air and concrete. Consequently, plastic shrinkage may occur even if the ambient temperature is not particularly high. A stiff breeze coupled with low humidity and moderate temperatures can create a major problem.

Concrete temperature is a function of the specific heat, weight, and temperature of the individual mix ingredients (Designation: 35). All concrete temperatures are important, but particularly with mass concrete because of its potential for thermal cracking. To minimize this potential, placing temperatures are restricted to 50°-60°F. Thermal cracking results from a steep temperature gradient from the interior of the mass to the exterior. If you start with a high placing temperature and add the heat generated by hydration, the interior temperature can get considerably higher than the exterior which is primarily influenced by the ambient temperature. In structural concretes with thinner members, thermal cracking is not the concern, but instead is other quality control measures such as water requirement, durability, and strength development. Since placing temperatures are a concern, many times there is a need to reduce the temperature of one or more ingredients. The most expedient means of reducing concrete temperature is by reducing aggregate temperatures, but this involves a sizeable investment in equipment. The easiest way to achieve a temperature reduction is by chilling the mix water. Normally a 4°F water reduction will reduce the concrete temperature 1°F. If necessary, ice may be substituted for water on a pound for pound basis, but it must be chipped or flaked. Many people get concerned about cement temperatures and in some cases this is warranted, but the rule of thumb is: A 10°F cement temperature reduction will reduce the concrete temperature 1°F. There is no standard specification for maximum cement temperature, so it must be specified by the user. At Auburn for example, both cement and pozzolan will have maximum delivery temperatures of 150°F.

The time-of-set influences the strength development and is dependent upon one or more of the following: cement type, concrete temperature, slump, curing temperature, and the type of chemical admixture used. If concrete sets too slowly, it affects timely finishing, form removal, and other scheduling. If concrete sets too rapidly, it may preclude proper finishing and reduce ultimate strength.

As you can see, there are a number of properties of fresh concrete that are important and do affect properties of hardened concrete. To conclude this discussion, I will briefly outline the testing procedures used to measure the fresh concrete properties. Obviously, the first step and most essential part of testing is sampling which is discussed in Designation 2. Quality evaluations are based on tests of a very small percentage of the concrete placed; therefore, each test must be performed on a sample that is representative of all the concrete, not just that portion that looks good. Unless proper precautions are exercised during sampling, the test results are essentially worthless.

Workability is somewhat like the weather; it is often discussed, but nobody is able to test for it. There are no standard field tests which quantify this property. However, there have been several laboratory tests and one of the first ones was developed by T. C. Powers in 1932. Mr. Powers said that workability should be a measure of the amount of work done to change the shape or remold a concrete sample. His test used a flow table set for 1/4-inch drops and measured the time necessary to remold a truncated cone shape into a cylindrical shape. There used to be a remolding apparatus here in the laboratory, but it has not been used in several years.

A few years later, in 1940, a refinement of Powers' test was made in Sweden. In this new development the flow table was replaced with a vibrating table. The Vebe or V-B test was also named after its developer and is primarily a European test. The vibrating table was selected because of the similarities to actual consolidation of the concrete in the field. This is a good test, but its range of application is limited to dry mixes with less than a 1-inch slump.

Another widely used workability test is the compacting factor test. This test was developed in England and measures the work done in an inverse manner; i.e., it measures the degree of compaction achieved by a standard amount of work. The compacting factor reflects the relative density of the concrete in a standard mold after it has been dropped from a standard height. This is another test designed for dry mixes which cannot be defined by the slump test. For these mixes, it is more sensitive and consistent than the slump test.

While discussing workability tests, references to the slump test are inevitable, even though the slump test is a measure of consistency and not workability. The slump test (Designation 22) is a very common test and is used in both the laboratory and the field. This is a very simple test and the apparatus is strong, portable, and relatively inexpensive. Although it has limitations, the slump is useful in controlling concrete by detecting differences in water content of intended identical mixes. For most of your concrete testing (1/2 to 6 inches of slump) the slump test is applicable, but with very stiff mixes such as those used in pipe manufacturing, the slump test shows no variation between the mixes. These mixes are called no- or zero-slump concrete. Another problem with the slump test occurs with lean, harsh mixes. These concretes often have uneven subsidence which means that slumps vary widely, even with the same mix and it is impossible to determine exactly where to measure the slump. Designation 22 requires an average slump, but ASTM requires a measurement at the original center of the sample. If the concrete lacks plasticity or cohesiveness, generally the slump will shear and the test should be repeated or the mix redesigned. There are other consistency tests, but first let me correlate values for the tests just mentioned.

Consistency description	Slump, in.	Vebe, s	Compacting factor
Extremely dry	-	18-32	-
Very stiff	-	10-18	0.70
Stiff	0-1	5-10	0.75
Stiff plastic	1-3	3-5	0.85
Plastic	3-5	0-3	0.90
Flowing	5-7	-	0.95

Another consistency test commonly used in California is the Ball-penetration. In 1950 J. W. Kelly developed this as an alternative to the slump test. This test requires a larger concrete sample, but it is much simpler and quicker than the slump test. An added benefit is that it can be used for concrete already in place. The ratio of slump to penetration varies from 1.5 to 2.0. Penetration is fairly constant for a given mix. This test has been standardized by ASTM and has been used on a limited basis.

A new consistency test that has been developed within the past few years is the K slump. This is another simple, quick test, but the small sample size creates an inherent variability greater than that associated with the slump test. Consequently, it has not been adopted by ASTM, but it is currently under review.

Both of the primary performance requirements for concrete; viz, strength and durability, require long-term testing. Since both of these properties depend upon the water-cement ratio, there is a need for a rapid determination of the actual water and cement contents. Unsuccessful attempts have been made with nuclear instruments and electrical conductivity equipment. Currently, there is a method that holds some promise. The Rapid Analysis Machine was developed in England and has been modified somewhat after some research at the Corps of Engineers' Construction Engineering Research Laboratory. In this country the test is known as the Kelly-Vail method. It requires 10-15 minutes to perform the analysis. Briefly, a 16-20-pound sample is washed over a No. 100 sieve to remove the fines which represent the cement content. You must make corrections for any aggregate or admixture particles in the material passing the sieve. Laboratory test results have shown there is a 95 percent confidence of measuring the cement content within 25 lb/yd^3. The English have made this test a part of their standards and testing evaluations are being conducted in this country. Water contents can also be determined fairly accurately by drying the sample.

Actual or yield mix proportions are most commonly derived from results of the unit weight test (Designation 23). The unit weight or density is the actual weight of concrete in a given volume. You will make several yield calculations during the next few days.

The volume of voids or air content is obtained by one or more of the following methods: pressure, gravimetric, or volumetric. The pressure method has been adopted throughout the Bureau and is described in Designation 24. Basically you pump up a volume of air to a predetermined pressure and allow it to expand into a container of fresh concrete. This compresses the air voids in the concrete and the reduction in pressure can be calibrated to give you directly the percent air in the concrete. The gravimetric air is a value determined during the yield calculations. You will become more familiar with this procedure throughout the week. The last method mentioned above was the volumetric. This procedure is more cumbersome than the pressure method and uses a much smaller sample (0.25 ft^3 versus 0.075 ft^3). The method is prescribed by ASTM and measures the air content by displacement with water. Although the pressure method can be used to determine air contents of lightweight concrete, the volumetric, or Roll-A-Meter method is faster and probably more accurate. In addition to the Roll-A-Meter, the Chace Air Indicator is a volumetric test for air contents. This test uses only the mortar portion of the concrete and measures the air content by displacement with alcohol. As the name implies, this is only an indication and should be used only for check tests in conjunction with a standard meter. Generally, the method is considered accurate within 0.5 percent.

Usually bleeding is measured relative to another mix and has no value in itself. Procedures are described in Designation 25 and are very similar to ASTM requirements. Essentially the volume of bleed water is measured and then expressed as a percentage of the total mix water.

Plastic shrinkage can affect the performance of hardened concrete through appearance and in some cases by increasing the permeability; however, there are no standard tests for measuring the amount of volume change. The important thing to keep in mind though is that keeping the concrete surface moist is an easy means of preventing plastic shrinkage.

It would seem simple enough to measure concrete temperatures by inserting a thermometer into the matrix, but we currently have a contractor that disputes that thesis. This case will be discussed more thoroughly next week. Unless you hear something to the contrary, we will continue to measure concrete temperatures with a thermometer in the matrix.

Time-of-set determinations are normally made in the laboratory and involve some form of penetration resistance. Our current procedures follow ASTM and only use the mortar portion of the concrete. Arbitrary limits of penetration resistance of 500 and 4,000 lb/in^2 have been defined as initial and final set, respectively.

You can readily see that the properties of fresh concrete do have an impact on hardened concrete. The prescribed test methods are important and should be performed properly and accurately - the results are very important to quality control.

BUREAU OF RECLAMATION
DIVISION OF RESEARCH AND LABORATORY SERVICES
CONCRETE AND STRUCTURAL BRANCH

STRENGTH PROPERTIES OF HARDENED CONCRETE

By

J. R. Graham*

Introduction

Strength is but one of several mechanical properties of hardened concrete. Stiffness, plasticity, elasticity, and energy capacity are some others of considerable importance. Time will require that we only discuss strength in this session.

When we speak of strength of concrete we are referring to a very complex property of a complex heterogeneous mixture of solids and gel. The strength of concrete therefore is derived from a combination of physical and chemical bonds in the matrix which is encapsulating the aggregate. It appears that in general, the strength of concrete made with aggregate of adequate strength is governed by the strength of either the cement paste or of the bond between the paste and the aggregate particles.1/**

In a broad sense, strength refers to the ability of a structure or material to resist loads without failure which may occur by rupture due to excessive stress or may take place owing to excessive deformation. Brittle materials like concrete generally fail by rupture whenever one of the principal stresses reaches the ultimate strain as determined by a simple tensile or compressive test.

There are several reasons why the strength of concrete must be known. First, concrete structural design traditionally has been based upon the use of allowable stresses which are percentages of the ultimate strength of the concrete. More contemporary design approaches use the ultimate strength of the concrete as a design basis. In the case of older concrete structures, the safety of the structure may be ascertained, in part, by knowing the present strength of the concrete in situ. The tensile strength of concrete and bonded joints is a mechanical property that is needed and used by designers in their structural earthquake analyses. It is also important to know what the actual properties of the in situ concrete in a structure are many years after its completion because such data are used in longtime structural behavior studies.

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**Numbers refer to references at end of text.

Construction engineers and inspectors are vitally interested in the strength properties of concrete. It is their responsibility to assure that the in situ strength of the concrete is in accordance with the design strength and the specifications.

When one refers to the strength of concrete he, whether aware of it or not, will be talking about either actual or apparent strength. Too often we talk about the measured strength of a 6- by 12-inch (15.2- by 30.5-cm) cylinder, for example, and assume that its strength reflects the strength of the concrete in the structure. I like to refer to the measured strength of a test specimen as apparent strength and that of the in situ concrete as actual strength. Following this concept, then, when a test cylinder is broken, we determine an apparent strength. If we have followed accepted test procedures such as ASTM: C 39-72, and using our experience, it is possible to correlate that apparent strength to the actual strength in the structure.

The apparent strength of concrete is generally determined by applying a known load to a concrete test specimen of known size until failure occurs. The method of loading is the most common basis for designating or classifying mechanical tests. For example, there are five primary types of loading which governs the stress condition to be induced: tension, compression, direct shear, torsion, and flexure. Although these loadings can and do occur simultaneously in the actual structure other than in the standard triaxial test, they are generally applied independently to the test specimen. These are generally considered as destructive tests. There are also nondestructuve tests that are used in some cases to give a qualitative measure of concrete strength. A discussion of the various test methods is given in the next section.

Mechanical Properties Tests

Nondestructive Tests

There are several types of nondestructive tests used to evaluate concrete but only three will be mentioned here. The impact hammer, often referred to as the Schmidt Hammer, is used quite widely for evaluating the quality and/or strength of concrete. We consider it a very useful instrument for providing data with which to make a qualitative analysis of the concrete but not very reliable for use in making a quantitative strength analysis. It is lightweight, portable, and easy for one person to use.

The Windsor Probe is another so-called nondestructive concrete testing device that is sometimes used. The concept used by this device is to shoot a projectile into the concrete using a constant energy input. The depth of penetration is a measure of the strength and/or quality of the concrete. Since many factors such as age of concrete, maximum size aggregate, etc., affect the test results, we consider this testing device also valuable only for qualitative analyses.

The sonic velocity technique is another nondestructive testing tool useful for qualitatively monitoring concrete. This system, as with the others, is not useful for quantitatively evaluating concrete strength. Although primarily used in the laboratory, it can be used in the field to test in situ concrete. Simply stated, this system employs a sound wave which is transmitted through the concrete and the travel time and distance are recorded. A calculation is made to determine the velocity of the wave through the concrete. This velocity is a measure of concrete quality.

Destructive Tests

Compressive strength tests are by far the most commonly used method of determining concrete quality both in the field and in the laboratory. In the compressive strength test, an attempt is made to apply an axial load to a test specimen so as to obtain uniform distribution of stress over the critical cross section. Specimens for these tests are usually cylinders or cubes. Sizes range from 2 inches (5.1 cm) in diameter to about 22 inches (33.9 cm) in diameter. Lengths of cylinders are usually twice those of the diameter. The ultimate apparent compressive strength of concrete is obtained by dividing the total load applied to the specimen by the cross sectional area at the break.

Because of present-day design and construction concepts, particularly regarding earthquake loadings and temperature control, the need for tensile strength of mass concrete has developed. The tensile strength of concrete can be obtained by many direct and indirect methods including direct load, pullout, splitting, and some less proven methods, such as the ring test and the two-punch test. The most commonly used specimen is the cylindrical-shaped core or laboratory cast sample. All of the available methods have some very desirable properties, and all have some undesirable properties. The direct tension method is used most often here in the laboratories for several reasons:

1. It most closely approximates the actual tensile stress in the concrete, i.e., a tensile load is applied to get a tensile strength value. Other methods such as the splitting tensile test uses a compressive load to get a tensile strength.

2. It lends itself to the testing of cores, cylinders, etc. Great care must be taken when using this method to minimize excentricity of loading.

Other destructive tests which are used to determine certain properties of concrete include triaxial, shear, flexure, and torsion. Although these tests are used less frequently than those described, they do provide valuable data for certain design and construction needs. There are also many specialized tests conducted on concrete to evaluate a particular property under a given external loading and environment.

Factors Affecting Actual Strength of Concrete

You are undoubtedly aware that many factors affect, either positively or negatively, the actual strength of concrete. First, the ingredients used in the mix affect concrete strength development, quality and quantity of materials used, additives used such as air-entraining agents and water reducing-set retarding agents, and cementing materials used such as type of cement, pozzolan, etc.

The water-cement ratio used in the fresh mix has the single most significant bearing upon the ultimate strength of concrete. The use of high water-cement ratios is not only economically unattractive but can seriously impair the density of the concrete, its durability, and its strength. Table 16 on page 136 of the Eighth Edition of the Bureau's Concrete Manual shows the compressive strengths that are developed when different water-cement ratios are used. Figures 20, 21, 23, 31, and 29 in the Eighth Edition show the effects of cement content, void ratio, cement type, $CaCl_2$, and MSA, respectively, on strength.

The placement temperature of the concrete and the type and length of cure greatly affect its strength development. Concrete placed and cured under low temperatures develops strength considerably slower than that subjected to normal temperatures or steam curing, for instance. Steam curing is used frequently particularly in the prestress industry, to accelerate rate of strength development for early form removal and handling of precast elements.

Curing is the process of maintaining a satisfactory moisture content and a favorable temperature in concrete during hydration of the cementitious materials so that the desired properties of the concrete are developed.3/ You who are responsible for the concrete in new construction will be well aware of strength development since you will be required to get proper strengths through test cylinders. Those of you who are responsible for the operations and maintenance of the structure for many years will be faced with living with the durability of the concrete. Both properties are very important and the propriety of the curing sequence will have distinct impact on them. Figures 9, 10, 11, 30, and 173 in the Eighth Edition of the Concrete Manual show the effects of various curing conditions on the strength development of concrete.

Many other factors also affect the actual strength of concrete, including consolidation, aggregate quality and grading, and age. Regarding the latter, we have data to indicate that concrete can continue to increase in strength to 22 years' age.4/ Along with age, however, goes the possibility of deterioration due to internal or external forces.

Factors Affecting Apparent Strength of Concrete

To the laymen, the strength of concrete in a structure is the same as that measured by testing a particular size and shape specimen. In reality many factors can affect the apparent strength of concrete. Furthermore, for test results to be useful to designers and others, they must be presented in terms of standards and units familiar to the user. For instance, if the compressive strength of some concrete was determined by tests on cubes when in standard practice cylinders were regularly used, the results probably would have to be adjusted to correspond to those which would have been obtained from tests on the more commonly used cylinders. Unfortunately, because concrete is a nonhomogeneous material, all technologists do not agree upon the same conversion factors.

The size and shape of a test specimen have a very marked affect on the apparent strength of the concrete. Figure 227 on page 374 of the Eighth Edition of the Concrete Manual shows the effects of test cylinder size on the compressive strength of concrete. For example, a 2-inch (15.1-cm) diameter test cylinder will have an indicated strength 108 percent of that of a comparable 6-inch (15.2-cm) diameter cylinder. The indicated compressive strength of cubes is generally greater than that of a comparable cylinder. The height to diameter ratio of test cylinders has a great affect on the indicated strength of the concrete in the specimen. The ASTM: C 192-76 requires that the height of a cylindrical specimen be twice that of its diameter. Cylinders with h/d ratios other than two can be tested, but a correction factor must be applied to correct the test results to correspond to those obtained from tests on a specimen with h/d equal to two. This requirement for a specific h/d ratio is not an arbitrary requirement. There is a complex interaction between the concrete test specimen and the steel platens of the testing machine. This is commonly referred to as "end effect." The magnitude of the shearing stress induced by the end effect decreases and the lateral expansion increases with an increase in distance from the end faces. The end effect becomes negligible at a distance of about $\sqrt{3}/2$ times the maximum lateral dimension of the specimen as evidenced by the presence of relatively undamaged cones of this height in a standard test carried on to destruction.2/

Testing methods also greatly affect the apparent strength of concrete. For example, the apparent tensile strength of concrete varies dependupon whether the testing was done by the direct pull system, the splitting system, the two-punch system, or whatever. For example, investigators throughout the country, yes even the world, might agree only that the ratio of direct tensile strength to splitting tensile strength of concrete is between 0.4 and 1.0.

Specimen fabrication, preparation, and load rate of testing can substantially affect the apparent strength of the concrete. The use of ASTM standard procedures in the preparation and testing of specimens will go a long way toward minimizing variations in test results due to the mechanics of collecting the data. A testing laboratory must first be a precision organization then a production organization.

Relationships Between Strength Properties

There is no fixed relationship between compressive and tensile strengths of concrete other than the fact that concrete showing an increase or decrease in compressive strength will generally show a corresponding change in tensile strength. The ratio of tensile strength to compressive strength generally runs from about 0.03 to 0.1 with that of mass concrete being closer to the lower end of the scale. Large cobbles in mass concrete apparently have a marked affect on reducing the tensile strength while having less affect on the compressive strength.

It is very difficult to determine a particular and consistent relationship between tensile strength values obtained from different test methods such as splitting tensile and direct tensile. About the only thing that can be said with any certainty is that indicated tensile strength values obtained by splitting methods are usually higher than those obtained on comparable concrete by direct tensile methods. This ratio, as previously mentioned, varies from 0.4 to 1.0. Many of the other variables previously discussed such as specimen size, aggregate size, etc., greatly affect the relationship between test method results.

In summary, you who will have responsibility for determining the compressive strength of concrete for whatever purpose will want to be sure that every step of the operation is done strictly according to pertinent specifications and standards. If you do so then those who use the apparent strength data that you have obtained will be able to relate it confidently to the actual in situ concrete strength.

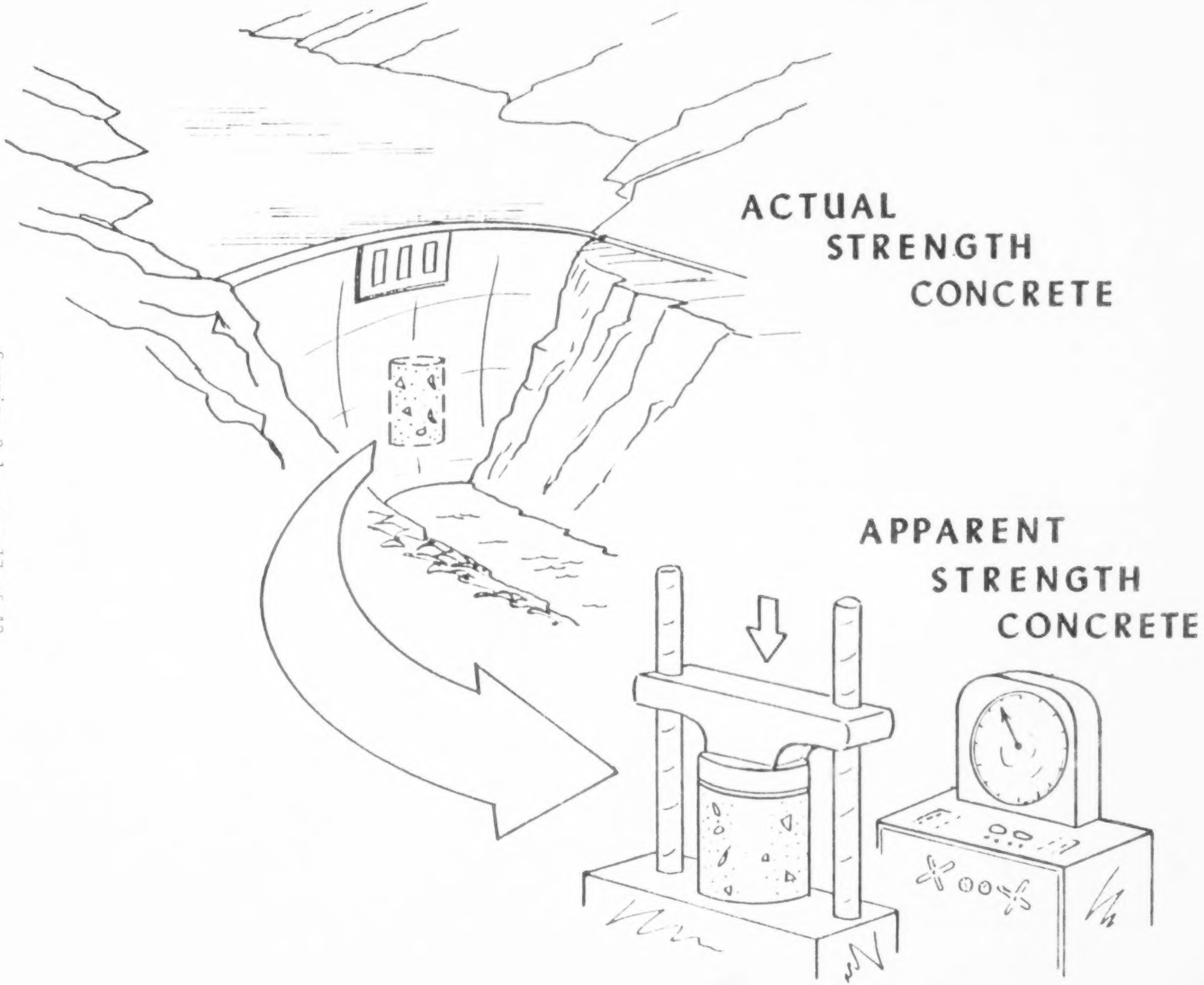
References

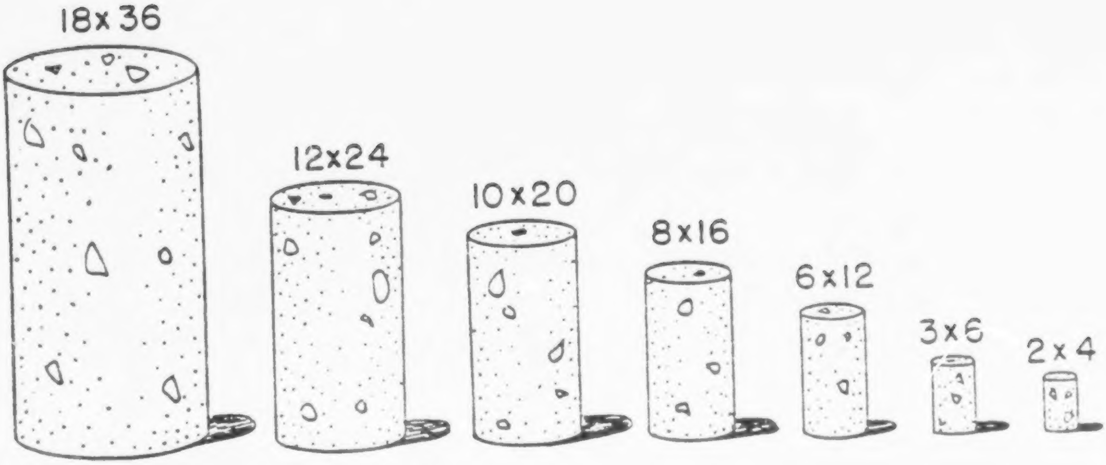
1. Concrete Manual, Bureau of Reclamation, Eighth Edition, Revised Reprint, 1981.

2. Neville, A. M., "Hardened Concrete: Physical and Mechanical Aspects," American Concrete Institute, Monograph No. 6, 1971

3. ACI Standard Recommended Practice for Curing Concrete (ACI 3-08-71), American Concrete Institute Manual of Concrete Practice, Part I, 1974

4. Higginson, E. C., Townsend, C. L., "Properties of Mass Concrete to 22 Years' Age and Their Influence on the Design of Large Concrete Dams." Prepared for the International Commission on Large Dams, Eighth Congress on Large Dams, Edinburgh, Scotland, 1964


ACTUAL STRENGTH CONCRETE
APPARENT STRENGTH CONCRETE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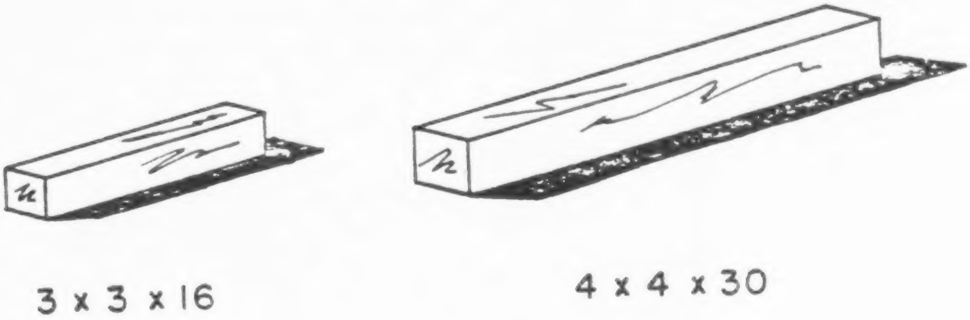
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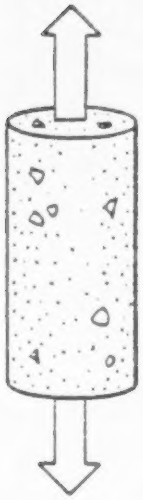
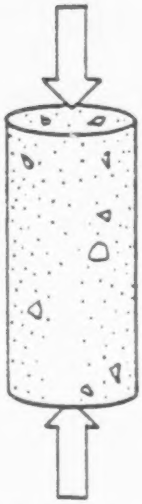
PRISMS

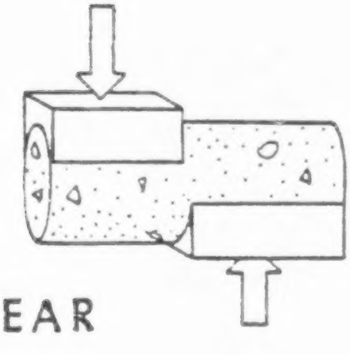
LOADING METHODS



TENSION



COMPRESSION



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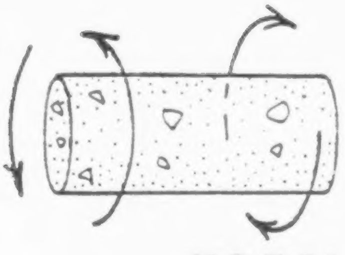
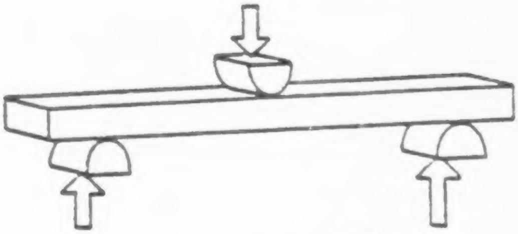
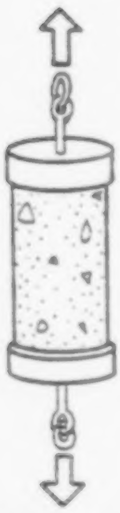


TORSION

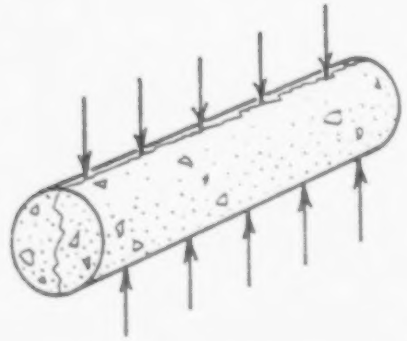


FLEXURE

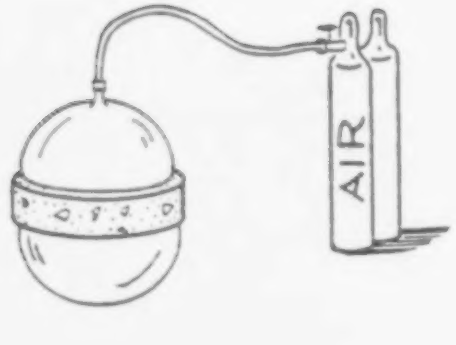
TENSILE STRENGTH TESTS



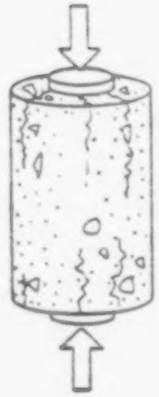
DIRECT TENSION



SPLITTING TENSION



RING TENSION



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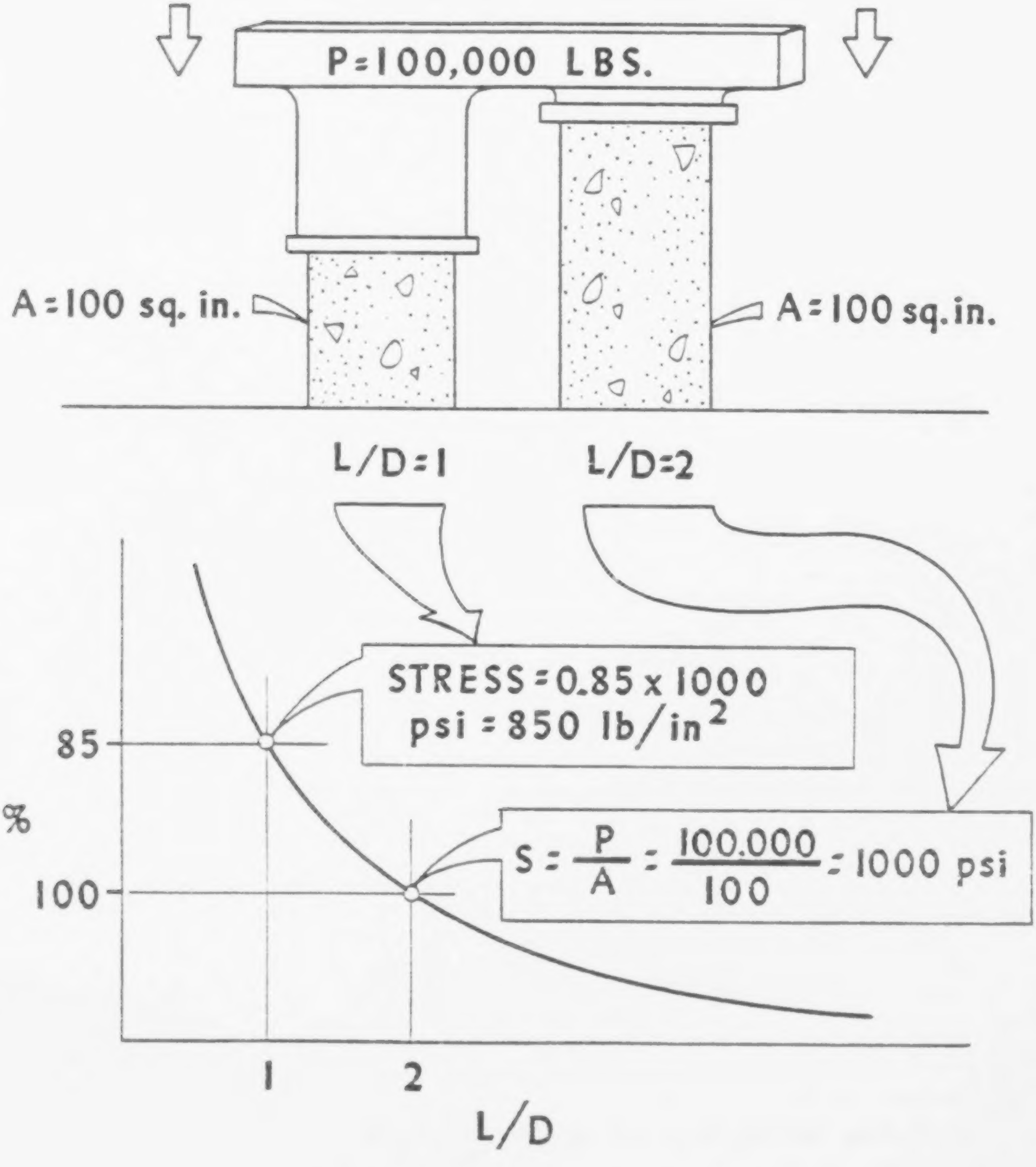
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A=100 sq.in.
L/D=1
L/D=2
STRESS=0.85 x 1000 psi = 850 lb/in²
S = P/A = 100.000/100 = 1000 psi
85
%
100
1
2
L/D

BUREAU OF RECLAMATION
DIVISION OF RESEARCH AND LABORATORY SERVICES
CONCRETE AND STRUCTURAL BRANCH

DURABILITY OF CONCRETE

by

R. N. Hess*

Good quality concrete must have adequate workability, strength, and durability. Durability is of primary importance for the concrete must withstand the effect of service conditions. Many destructive forces can affect concrete durability. These destructive forces may be either physical or chemical in nature.

Concrete is damaged when exposed to repeated cycles of freezing and thawing by the expansion which water undergoes upon freezing. This expansion of water when frozen amounts to about 9 percent of its liquid volume. If voids within the concrete are 91 percent filled with water, stresses may be created that can rupture the concrete. If these voids are filled with water beyond this critical point (i.e., the point where insufficient void space remains to contain the expansion of freezing water), harmful stresses will be developed. Concrete exposed to water long enough to become saturated prior to freezing is extremely susceptible to damage. Controllable factors influencing the resistance of concrete to this destructive force are (1) quality of materials, (2) water-cement ratio, (3) curing conditions, and (4) air-entrainment. Aggregate quality is the most significant variable encountered in the materials used in Bureau concrete. Freeze-thaw durability testing of concrete containing aggregates proposed for use is routinely included in aggregate suitability evaluations.

Concrete durability testing facilities at the Engineering and Research Center include two automatically controlled laboratory freeze-thaw test exposures and also an outside storage area where concrete is exposed to natural weathering. The standard freeze-thaw apparatus subjects concrete specimens to freezing and thawing in water. The specimens are 3- by 6-inch cylinders cast from concrete containing the test aggregate in a standard concrete mix (3/4-inch maximum size aggregate, 0.51 water-cement ratio, 6 percent entrained air, 3-inch slump). By standardizing concrete mix properties, any variation in durability can be attributed principally to the aggregates used. Some of the standard concrete specimens are also tested to determine compressive strength and modulus of elasticity.

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The freeze-thaw specimens are inserted in loose fitting rubber sheath containers, covered with water prior to testing, and then placed upright in storage racks. The test cylinders are subjected to alternate freezing at 10° F and thawing at 70° F. Calcium chloride brines (maintained at either 10° F or 70° F) are used as the freeze-thaw media. These brines are continuously pumped through two specimen storage cabinets so that specimens in one cabinet are freezing while specimens in the other cabinet are thawing. The warm and cold brine paths are interchanged each 1-1/2 hours. Specimens complete one cycle of freezing and thawing in 3 hours. Specimens are generally examined every 2 weeks for spalling or aggregate popouts and a weight loss determination. The failure criteria is a weight loss of 25 percent - concretes which withstand 500 cycles of freezing and thawing without failure are considered to have satisfactory durability.

The second controlled laboratory test exposure uses a "sub-zero" apparatus which freezes specimens in air and thaws them in water in a large, sealed watertight cabinet. Air circulated through freezing coils and around the test specimens is used as the freezing medium and water maintained at 50° F is used for thawing. Freeze-thaw cycles can be adjusted to a desired length of time from 1 to 23 hours. Various shapes and sizes of specimens can be tested in this apparatus. Specimen failure may be determined by visual examination, weight loss determination, or by a change in the modulus of elasticity.

Freezing and thawing deterioration may be controlled in several ways. Air entrainment in concrete (produced by the addition of a suitable admixture during mixing operations) contributes more to freeze-thaw durability than any other single factor. Durability may be increased two to four times by optimum entrained air contents. The air voids are formed within the mortar portion of the concrete and to effectively relieve the stresses developed within concrete due to expansion of water while freezing, these entrained air voids must be small, well spaced, and closely grouped together. The use of good quality aggregate contributes to concrete durability. Freeze-thaw resistance is improved by reducing the water-cement ratio. Curing conditions also affect concrete durability. An adequate initial moist curing is essential to insure adequate strength development and watertightness before freeze-thaw exposure, but prolonged periods of drying following initial curing (which allows free water within the concrete to evaporate) often improves concrete durability.

In the geographical area covered by the Bureau's operations, protection against concrete damage caused by the chemical attack of sulfates (recognized as a problem since the discovery of portland cement in 1824) is of primary concern. These sulfates are found in the arid and semiarid regions throughout most of the 17 western United States. The sulfates of sodium, calcium, and magnesium which may cause serious damage to concrete are highly soluble and originate in soils and ground water. In areas where sufficient rainfall occurs, these sulfates are leached out and carried away by runoff and percolating ground water thus reducing the danger of sulfate attack.

Concrete resistance to sulfate attack depends upon the degree of cement paste chemical stability. The sulfates in soil and ground water are absorbed by concrete and react with certain compounds in hydrated cement paste. This chemical reaction results in a crystalline product that occupies a greater volume than that occupied by the original compounds. As this volume increase occurs, it causes tensile stresses that may exceed the tensile strength of concrete. The cement compounds which may be attacked by sulfates are tricalcium aluminate (C_3A) and calcium hydroxide. With tricalcium aluminate (present in most cements in concentrations ranging from 1 to 15 percent) the reaction progresses at a fairly rapid rate with pronounced expansion at an early age. This reaction ends with the formation of a sulfoaluminate crystal (sometimes called "ettringite") which occupies about 2-1/2 times more volume than the originating compounds. With the reaction involving sulfates and calcium hydroxide (or free lime) the formation of calcium sulfate takes place at a slower rate but will also eventually cause concrete damage.

Many factors determine the rate and completion of these reactions. It is possible that due to moisture conditions sulfates may be present only for short, infrequent periods of time, and if the concrete is very dense and impermeable, the sulfate attack rate is very slow. However, if sulfate concentrations are high with sufficient water available to put it in solution and a poorly resistant cement is used, concrete failure can rapidly occur. The most important factor in determining the rate and degree of sulfate attack are the sulfate concentrations in adjacent soil and ground water and the type of cement used. Soil moisture is required to put the sulfates in solution, and since most Bureau concrete is used for hydraulic structures, excess water is usually present due to seepage and irrigation operations.

Sulfate attack investigations may include testing of the water used in mixing and curing of concrete or in aggregate washing operations and also the testing of soil or water which will be in contact with concrete to determine harmful sulfate concentrations. Three laboratory test exposures are used at the Engineering and Research Center to evaluate the sulfate attack resistance of various cements, concretes, and admixtures. The standard test specimen is a 3- by 6-inch concrete cylinder with inserts cast in the cylinder ends to permit expansion measurements. One test is an accelerated one in which the concrete specimens are subjected to a cycle consisting of a 16-hour soaking period at 73.4° F in a 2.1 percent sodium sulfate solution followed by an 8-hour drying period at 130° F. The other tests involve continuous soaking of the specimens in either a 2.1 or a 10 percent sodium sulfate solution at room temperature. The accelerated and 10 percent continuous soaking tests produce failure in about one-sixth the time required by the long time "standard" 2.1 percent continuous soaking test. Test specimens are evaluated periodically by visual examination and expansion measurements. Expansion of 0.5 percent is considered as failure in these tests. These tests often require many years for completion and are used to compare the relative resistance of various types of cement or admixtures and to evaluate new products as they are developed.

None of the several methods used to reduce the effect of sulfate attack provide complete protection, but they will reduce the deterioration rate to an acceptable level. In the case of fairly massive structures, sulfates do not present a problem because of the inaccessibility to interior concrete by sulfate waters. The use of special sulfate resistant cement, such as a type II or type V, is the most common method of preventing sulfate attack. These cements limit the maximum allowable percentage of C_3A (tricalcium aluminate), the main cause of sulfate deterioration, and are very resistant to sulfate attack. Type II cement limits C_3A content to a maximum of 8 percent and is used where moderate sulfate attack may occur - type V cement limits C_3A content to a maximum of 5 percent and is used where severe sulfate attack may occur. The use of extra cement, which improves concrete strength and watertightness, increases sulfate resistance. Use of air entraining agents to increase workability and consolidation of concrete also improves sulfate resistance. Some pozzolans, which react with free lime to form stable calcium silicates, reduce the danger of sulfate attack but other pozzolans may reduce sulfate attack resistance. Therefore, pozzolans must be tested to determine the effectiveness of their sulfate resistance properties. Maximum protection against sulfate attack is provided with type V cement and a pozzolan determined by tests to improve sulfate resistance. In extreme cases where sulfate concentrations are very high, protective coatings may be required to assure adequate protection. Often a combination of these various methods is necessary.

Another type of chemical attack concerns cement alkali-aggregate reactivity. For many years concrete aggregates were considered to be inert materials but in the late 1930's, the California State Highway Department when confronted with an unusual type of concrete failure caused by excessive expansion and cracking of concrete structures, determined that certain aggregate types were responsible for these failures. In 1940, Mr. T. E. Stanton of the California Highway Department first reported that this deterioration was caused by an expansive reaction between certain aggregate constituents and the alkalies in cement. These alkalies (sodium and potassium oxides) are minor cement constituents and generally comprise less than 1 percent of the cement. At this time concrete deterioration in numerous structures caused by the suspected cement alkali-aggregate reaction was confirmed.

Parker Dam, a Bureau structure completed on the Colorado River in 1938, was among those affected by this reaction. Aggregates of satisfactory physical quality were used at Parker Dam, but within 2 years after completion concrete surfaces above the waterline began to develop extensive and severe cracking. Three of the four brands of cement used at Parker Dam had alkali contents ranging from 1.13 to 1.42 percent. In 1941, a conference attended by Bureau personnel, cement manufacturers, research chemists, and other interested organizations was held in Denver to discuss this problem and Parker Dam was a principal topic. Intensive studies were begun at Parker Dam and in the Bureau and other laboratories throughout the country. Field studies which included expansion measurements of cracks, drill core studies to determine cracking depth and measurement of interior expansion, and petrographic examination of cores provided considerable information about the alkali-aggregate reaction. Numerous other structures, some of which had used the same aggregate source as Parker Dam, were also examined and no deterioration was noted in any concrete made with cements containing less than 0.60 percent alkali. In addition to field studies, many laboratory investigations were made to study the reaction, determine the rock types involved, find means to inhibit the reaction, and develop tests to identify the harmful cement-aggregate combinations. A discovery by Mr. Stanton that the alkali-aggregate expansion could be laboratory produced when mortar bars were stored for a period of time in a sealed, moist bucket led to the development of a test of mortar bars with gage points to permit length change measurement. Literally hundreds of cement-aggregate combinations were compared by determining expansions produced in mortar bar specimens stored in the presence of water in a sealed container. As a result of these studies, petrographic examination can with reasonable accuracy establish potential aggregate reactivity based upon the reactivity of identified rock types. All these studies indicated that cements with a total alkali content below 0.60 percent (expressed as soda equivalent) cause little expansion with reactive aggregates. Such cements are termed "low-alkali" cements. "High-alkali" cements have a total alkali content greater than 0.60 percent (expressed as soda equivalent). The soda equivalent is computed as the total percentage of sodium oxide (Na_2O) plus 0.658 of the percentage of potassium oxide (K_2O) - i.e., each percent of K_2O is chemically equivalent to 0.658 percent Na_2O.

The alkali-aggregate concrete deterioration is caused by the formation of strong caustic solutions during mixing, the period of set, and subsequent curing period as water dissolves significant proportions of alkalies from the cement. This alkaline solution concentration increases as the calcium silicate and aluminate progressively hydrate and extract water from the alkaline solution. Susceptible aggregate particles are attacked by the caustic liquids formed in the concrete and alkali-silicate gels form from certain types of silica in the rock materials. These alkali-silicate gels (which have a great affinity for water) are deposited in the pores surrounding reactive rock particles. After the concrete has set and later becomes moistened, the water is absorbed by the gel which swells and builds up pressures which produce stresses ultimately sufficient to rupture the cement paste. Wide variations in the rate and degree of deterioration are caused by the reactivity of aggregates, cement alkali content, available moisture, and temperature range.

The standard mortar bar test to evaluate aggregate reactivity uses mortars made with both high- and low-alkali cements. Mortar mix proportions (by weight) are one part cement to 2.25 parts sand - coarse aggregates (when tested) are crushed into sand sizes. The mortars are fabricated into 1- by 1- by 11-1/4-inch length change specimens, fog cured for 7 days, and then stored upright in sealed containers in the presence of but not in contact with water and maintained at a temperature of 100° F. Length change measurements are made at regular intervals. Expansion in the high-alkali cement mortar in excess of 0.1 percent in 6 months or 0.2 percent in 1 year indicate potentially reactive aggregate requiring the use of low-alkali cements. Expansion measurements of the low-alkali cement mortar bars are made to confirm that low-alkali cements will control reactive expansion. An interesting phenomenon related to alkali-aggregate reaction is a "pessimum" condition sometimes encountered when extremely reactive aggregates are combined with high-alkali cement. Small amounts of these reactive aggregates blended with a neutral nonreactive aggregate may produce high expnasion; however, increasing the amount of highly reactive aggregate blended with correspondingly smaller proportions of neutral aggregate sometimes reduces expansion and frequently 100 percent of the highly reactive aggregate produces almost no expansion. The "pessimum" condition is the amount (less than 100 percent) of highly reactive aggregate which is most harmful when combined with high-alkali cement. As petrographic analysis of aggregates can generally indicate reactive aggregates which require the use of low-alkali cement, mortar bar tests are now usually made only when questionable amounts of known reactive minerals are present in the aggregate.

Another type of chemical reaction in conrete is known as "cement-aggregate" reaction or "Kansas-Nebraska" type reaction. This Kansas-Nebraska or K-N reaction occurs in Kansas, Nebraska, and parts of eastern Colorado and Wyoming. In this area coarse aggregate larger than about 3/4-inch maximum size is almost nonexistent. When used in concrete, locally available aggregate in this area may produce large expansions resulting from the reaction between aggregate and cement, and this reaction can not be controlled by the use of low-alkali cement alone. Extensive laboratory tests have shown that the use of crushed limestone coarse aggregate (which acts as a "sweetener") in conjunction with low-alkali cement and natural sands is required to produce durable concrete in this area. Another peculiarity of the K-N type reactive aggregates is that their reactive potential may not be revealed by the standard mortar bar tests. A special test developed to evaluate this K-N reaction subjects mortar bars to special curing consisting of alternate periods of soaking, drying, heating and cooling, followed by storage in distilled water at 73.4° F. However, petrographic examination can usually identify the aggregates responsible for the K-N reaction, and therefore it is a Bureau practice throughout wide areas of the midwest to specify the use of limestone coarse aggregate with all natural sands from certain river drainage areas, including the Republican, Loup, Kansas, and Arkansas Rivers and portions of the Platte River.

The principal methods for control of alkali-aggregate reactivity include (1) use of nonreactive aggregates, (2) use of low-alkali cement with reactive aggregates, (3) use of certain pozzolans, and (4) use of sweetening limestone coarse aggregate with low-alkali cement in conjunction with natural sands from the "Kansas-Nebraska" area.

Although other physical forces or various chemical agents may shorten the useful life of concrete structures, this brief survey has considered only the most common durability problems encountered in Bureau construction. Concrete durability is of primary importance. It is not sufficient to only produce high-strength concrete. Although durable concretes will usually have ample strength, strong concretes are not always durable. It is necessary to use the best quality and most suitable materials for concrete as required by anticipated service conditions. The problem of durability is further complicated by the fact that many of the destructive forces responsible for concrete damage cannot be completely eliminated. The principal means to produce a durable concrete that is subjected to some of these destructive forces are:

A. Freezing and Thawing

  1. Use good quality aggregates
  2. Use optimum amounts of entrained air
  3. Lower the water-cement ratio (0.45 W/C ratio for severe exposure)
  4. Proper curing

B. Sulfate Attack

  1. Use special sulfate-resistant cement
     a. Type II for moderate sulfate attack
     b. Type V for severe sulfate attack
  2. Use extra cement (one bag more per cubic yard of concrete)
  3. Use optimum amounts of entrained air
  4. Use certain pozzolans

C. Alkali-Aggregate Reactivity

  1. Use nonreactive aggregates
  2. Use low-alkali cement with reactive aggregates
  3. Use certain pozzolans
  4. Use sweetening limestone coarse aggregate with natural sands in the "Kansas-Nebraska" area

BUREAU OF RECLAMATION
DIVISION OF RESEARCH AND LABORATORY SERVICES
CONCRETE AND STRUCTURAL BRANCH

SPECIFICATIONS FOR CONCRETE

by

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J. L. Hart**

Introduction

In assuring quality concrete construction, concise specifications and our understanding and use of them are paramount. Poor concrete construction and contractual problems are often the result of poorly worded or wrongly interpreted specifications.

On one hand, specifications requirements should be uniform and consistent from one contract to another. For instance, if we allow the use of $CaCl_2$ in winter concrete for Tyzack Dam, we should allow it for other features involving outdoor concrete construction on the Central Utah Project. Otherwise, the contractor for the second contract may complain that we are being needlessly inconsistent. On the other hand, we must determine which specifications requirements are applicable for each job individually. For instance, we specified that type II cement be used in most of the Granite Reef Aqueduct. In reach 2, however, we required the use of Type V cement because we knew there were isolated areas containing high concentrations of sulfate in the soil. The dilemma then is "To change or not to change." Our position is that the construction specifications should vary whenever there is sound justification for change.

How Concrete Specifications are Written

The construction specifications for any particular contract are composed from "Guide Paragraphs" which cover each aspect of construction. Guide paragraphs for each section are maintained by Bureau people most knowledgeable in that subject. The Concrete and Structural Branch maintains the "Concrete Guide Paragraphs." In addition, we revise a working copy of these guides for each individual job and submit it to the specifications writers who then assemble all revised guide paragraphs into each construction specifications. Construction specifications for smaller jobs may be issued by the regional offices who also use these guide paragraphs. The effect of the guide paragraphs is to produce uniform specifications from contract to contract but to make the necessary changes for each individual job.

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There are three different sets of guide paragraphs for concrete specifications requirements:

1. Regular (concretes 1 through 26)
2. Short (concrete 28)
3. Very short (concrete 31)

As a general rule, the "Regular" guide paragraphs are used whenever there is a large amount of cast-in-place concrete. These paragraphs are very complete and specify in detail a quality-controlled concrete-producing operation. Past experience has shown that this type of specifications assures better concrete than does a "product type" of specifications which requires only that the final product meet certain requirements. After all, a requirement like durability is not easily judged immediately. For jobs requiring smaller quantities of concrete, ready-mix concrete will almost certainly be used and it becomes impractical to specify detailed concrete-producing operations such as finish screening and supplying of Government-tested cement. When the total amount of concrete is less than 300 yd^3, detailed specifications become impractical, and we then use the "Very Short" guide paragraphs which are basically a "product type" of specifications paragraphs. When the amount of concrete involved is between 300 and 1,000 yd^3 and where features of the concrete construction are such that ready-mix concrete is also likely to be used such as in transmission line work, the "Short" guide paragraphs are used. The "Short" paragraphs are a compromise between the "Regular" and "Very Short" paragraphs.

Do not get the mistaken impression that the quality of concrete becomes less important as the volume of it decreases because quite often the opposite is true. For instance, 2 yd^3 of poor quality concrete placed in Ellsworth Substation may have a greater effect than the same amount of poor-quality concrete in the center of the foundation of Havasu Pumping Plant. We make our requirements less stringent for smaller concrete quantities because of economic necessity, not because that concrete is any less important.

Changes in concreting technique and available machinery, increased technical knowledge, service histories of existing concrete structures, and discovery of confusing and misleading rhetoric make periodic revision of the concrete guide paragraphs necessary. Our last major revisions to the guide paragraphs were in 1984. In fact, the concrete paragraphs have been under a state of continuous revision since 1978, and change in concrete technology is continuing at a rapid pace. The input which we receive from Bureau field personnel, inspectors, and laboratory personnel alike is instrumental in showing us where changes are needed. We need your help to maintain efficient, realistic specifications requirements.

Specifications guide paragraphs are not to be confused with "Standard Specifications." Standard specifications are fairly complete specifications in themselves which deal with one phase of concrete construction and which do not normally change from one contract to the next. For instance, the Water and Power Resources Service "Standard Specifications for Concrete Curing Compound," dated October 1, 1980, is a separate document to which applicable construction specifications refer since there is usually no reason to vary requirements for curing compound from job to job. Although, standard specifications are also revised as necessary when the "state of the art" undergoes significant change. Major changes were made to the "Standard Specifications for Repair of Concrete," dated January 4, 1982, reflecting the advancements made in the maintenance and repair of concrete.

The Bureau of Reclamation Concrete Manual is also a part of the construction specifications. However, the contractor is liable for only those parts of the manual which are specifically referenced. In the event of conflict though, the construction specifications prevail. If, for instance, the Concrete Manual says that concrete may be cured by water curing and the construction specifications require curing compound, the contractor must use curing compound.

Many references are made within our construction specifications to pertinent ASTM (American Society for Testing and Materials) standards for materials and test methods. For instance, all concrete admixtures and cementitious materials must meet ASTM standards. Therefore, it is necessary for each construction office to have recent (not more than 2 years old) ASTM standards readily available for use by laboratory and inspection personnel.

Pertinent Aspects of the Guide Paragraphs

1. Composition (conc. 1). - Bureau specifications do not normally specify the mix proportions for concrete. The determination of the mix proportions is generally left up to the field engineering forces. However, the composition paragraphs do describe such important aspects as: maximum size aggregate, strength requirements, water-cement ratio, and slump.

It is Bureau practice to assure that concrete will be adequately durable over the years by specifying maximum water-cement ratio and maximum slump. We are able to do so because we pay separately for cementitious materials. Consequently, the contractor need not be concerned about total payment for cement and pozzolan and Bureau personnel can control the mix design.

On small jobs where the "Short" and "Very Short" guide paragraphs are used, we also specify minimum cement content or at least water-cement ratio to insure quality concrete and to aid the contractor in bidding since the cost of cement is usually included in the cost of the concrete. However, the Bureau's field personnel still have an option to control the mix if they choose to do so.

For large concrete dams, we generally include a tabulation in the specifications giving the contemplated ranges or limits for water-cement ratio, cement content, pozzolan content, and percentages of sand and each size of coarse aggregate. These ranges are included primarily for the benefit of bidders in determining their bid and are made possible by the mix design studies which we complete before specifications are issued.

2. Concrete Quality Control Measures and Concrete Quality Assurance Test Program (conc. 1.1). - These paragraphs discuss such things as: contractor's responsibility for quality control, Government's quality assurance program, methods of testing fresh and hardened concrete, and testing facilities to be provided by the contractor.

We believe that quality assurance of concrete is important enough to specify that the contractor provide sampling equipment and testing facilities for use by the Government as listed below:

(a) Central mix batch plant. -

(aa) An enclosed building of not less than 200 square feet adjacent to the batching and mixing plants, free from plant vibration and excessive plant noises, and furnished with all necessary utilities including lights, compressed air, water, room temperature control, and electrical power.

(bb) Mechanical sampling devices for safely procuring and handling representative test samples of aggregates and other concrete materials during batching and for obtaining representative concrete samples from a point in the discharge stream as the concrete is discharged from the mixers.

(cc) Ample and protected working space near the placement site and a means for safely procuring and handling representative concrete samples.

(b) Transit mix batchplant. -

(aa) Mechanical sampling devices for safely procuring and handling representative test samples of aggregates and other concrete materials during batching.

(bb) Ample and protected working space near the placement site and at the batch plant and a means for safely procuring and handling representative concrete samples at both locations.

The Government then obtains samples and conducts tests (with Government testing equipment) in accordance with the test methods specified. You should note that we now specify ANSI/ASTM Test Methods, wherever possible instead of Designations from the Concrete Manual. This is done as a convenience to the construction industry which usually is already following ASTM practices. This does not indicate a relaxing of our standards but rather the general improvement in ASTM Standards and an acceptance by this organization of our excellent test methods. In most

best copy available

cases now, there are very few real differences between the ANSI/ASTM Test Methods cited and the Concrete Manual Designations that they replaced.

3. Cement (conc. 2) and Cementitious Materials (conc. 3). - Selection, inspection and acceptance of cement and pozzolan are thoroughly covered here. We normally use Conc. 3 which specifies that acceptable pozzolans may be used to replace part of the cement in the mix. This allows the contractor more flexibility in obtaining cementitious materials according to his best interests while still meeting our needs.

The contractor is required to submit the following information to the project:

a. Names and addresses of plants from which cementitious materials will be shipped

b. Names and business addresses of companies supplying cementitious materials

c. Names and addresses of subcontractors to whom cementitious materials will be shipped if other than the prime contractor

d. The quantity of cementitious materials to be ordered

e. Whether or not cementitious materials will be bulk or in bags

f. Purchase order number or other means of identifying cementitious materials ordered by the contractor

When the contractor submits these data, they should be immediately forwarded to the Corps of Engineers in Vicksburg, Mississippi, so the cement and pozzolan test group will have adequate lead time to properly schedule testing.

Payment for cementitious materials is made separately from concrete. We usually pay the contractor for all cementitious materials used in batched concrete except for that in wasted concrete. It is the batch plant inspector's duty to determine, by means of batch count and average cementitious materials content per batch, the total amount used by the contractor. The price includes not only its purchase cost but also the cost of transportation, handling, and storing cementitious materials.

4. Admixtures (conc. 4). -

a. Set-accelerating admixtures (types C and E chemical admixtures). - Because of their corrosive properties, calcium chloride and other set-accelerating admixtures should not be used in concrete in which aluminum or galvanized metalwork is to be embedded or in prestressed concrete. Embedded metalwork is so plentiful in transmission line, substation, and switchyard work that it is common practice now not to use calcium chloride at all for that type of work. Recently, concern about the rapidly deteriorating metal in contact with concrete on Glen Canyon

Bridge has shed new light on the use of calcium chloride. Wherever minute electric currents are a natural phenomenon, the corrosive effect of calcium chloride is intensified.

Accelerators should not be used in concrete containing Type V cement as they generally reduce the resistance of concrete to sulfate attack. Otherwise, we allow the use of accelerating admixtures such as calcium chloride in concrete when the mean daily temperature falls below 41 °F for 2 out of 4 days prior to placement. The mean daily temperature is the average of the high and the low temperatures recorded in one 24-hour period.

b. Air-entraining admixtures. - The Bureau requires an air-entraining admixture (AEA) to be used in all cast-in-place concrete. The admixture used must conform to ASTM Designation: C 260. The specifications also requires that the admixture be added to the mixer in a portion of the mix water by means of mechanical batcher to help eliminate the chance of human error.

Air-entraining cements are not allowed because of their inability to entrain the proper amount of air under different conditions and circumstances. There are many variables from job to job that affect the amount of AEA required in a given mix and using an air-entraining cement would destroy the flexibility needed to properly control the entrained air content in concrete.

c. Water-reducing admixtures. - Specifications generally require use of a water-reducing admixture for all larger jobs. For smaller jobs, the use is optional and is left to the discretion of the contractor. The reason we do not require it on small jobs is that it involves handling another material and requires a separate dispenser so the cost involved may be greater than the savings from using it. Our specifications require that it conform to ASTM Designation: C 494. The cost of the admixture is not paid for separately but is included in the cost of concrete.

Sometimes WRA's and air-entraining admixtures are incompatible and will form as a solid precipitate if mixed together in water. Other times the effectiveness of the WRA is reduced because of an excess of silt and clay fines in the aggregate or because of a reaction with the cement. It is the contractor's responsibility to provide a WRA which is compatable with the materials being used and a waiver to discontinue using WRA should be discouraged.

d. Aluminum powder. - We specify the use of aluminum powder sometimes when a nonshrink concrete, mortar, or grout is needed to assure complete filling. Aluminum powder generates a gas that expands the concrete. If the forms restrain this expansion, the concrete is put into compression. Shrinkage still occurs but acts to release the compressive forces and so appears to be "nonshrink."

5. Water (conc. 5). - We specify that mixing water shall be free of objectionable quantities of silt and organic matter, but this is not very

definitive; in general, if it is muddy, it should be run through a settling pond so the silt settles out. It must not contain more than 3,000 mg/L of soluble sulfate. The best way to determine whether water is objectionably contaminated is to mix concrete using questionable water and using potable water. If the concrete's quality is reduced significantly by the water in question, the water should not be used.

6. Concrete Aggregate (conc. 6, sand; conc. 7, coarse aggregate; and conc. 8, production of sand and coarse aggregate). - Bureau specifications utilizing our regular guide paragraphs permit the contractor to obtain aggregate from any approved source. Usually this means that samples have been submitted to Denver for testing and approval of its quality. If a source has not been tested, samples from a contractor's proposed source should be sent in and tested before approval can be given. Occasionally, we will tentatively approve a source based on tests conducted by the Corps of Engineers or by State highway departments. On small jobs where the concrete is supplied by ready mix plants, the cost of testing the aggregate would exceed the bid price for the concrete. In these cases, we have tentatively approved a source on the word of the construction engineer that structures in the area containing concrete with this aggregate show no distress. Bear in mind that repair of concrete is very costly and that we can extend the service life of a concrete structure by using quality concrete aggregate.

a. Grading and free moisture. - The sand is required to be within specified limits for each sand size. These limits can be found on page 61 of the Eighth Edition of the Concrete Manual. We also specify a maximum limit of 6 percent free moisture in the sand and a limit on moisture variation. For mass concrete dams, we require three stockpiles during production of sand: one on which he is stockpiling his washed sand; one draining; and the third he is using from after it has drained for 48 hours so it has reached a fairly stable moisture content.

In the future, coarse aggregate for mass concrete will probably be separated in 3/16- to 1-inch, 1- to 2-inch, and 2- to 4-inch sizes. Coarse aggregate for structural concrete is generally separated in the following sizes: 3/16 to 3/4 inch, 3/4 to 1-1/2 inches, and 1-1/2 inches to 3 inches. Coarse aggregate for canal lining, however is separated into these sizes - 3/16 to 3/8 inch, 3/8 inch to 3/4 inch, 3/4 to 1-1/2 inches - because tighter control of the 3/16- to 3/8-inch size fraction is necessary to enable finishing machines to do the job properly. The maximum size specified depends mostly on placement dimensions, rebar configuration, and aggregate available for use.

We usually require finishing screening of each size of coarse aggregate over a vibrating screen at the batching plant. Material passing the 3/16-inch screen shall be wasted or can be used to make up deficiencies in the sand if introduced ahead of the sand classifier, but it cannot be dumped directly into the sandpile.

Coarse aggregate shall have, after finish screening, no significant oversize and not more than 2 percent significant undersize.

7. Batching (conc. 9). - Bureau specifications do not specify the type or size of batching equipment to be used. The contractor is required to provide, maintain, and operate equipment which will accurately determine and control the amount of each separate ingredient entering the concrete. The amounts of bulk cement, pozzolan, sand, and each size of coarse aggregate entering each batch of concrete shall be determined by weighing, and the amount of water shall be determined by weighing or by volumetric measurment. Admixtures may be batched by visual-mechanical volumetric measuring devices or by weight. Admixtures must not be added directly to the batch, but rather to the mixing water. The batching equipment is required to be constructed and operated so that the combined inaccuracies in feeding and measuring the materials do not exceed 1 percent for water, 1-1/2 percent for cementitious materials, 2 percent for sand and each size of coarse aggregate up to 1 1/2-inch nominal maximum size, and 3 percent for 3-inch nominal maximum size aggregate and admixtures. The scales and other measuring devices are required to be periodically checked for accuracy.

8. Mixing (conc. 10). -

a. The contractor is not limited as to type or size of batch mixer. Bureau specifications require that the mixer being used on the job be tested for adequacy of mixing by the Government by mixer performance tests in accordance with ANSI/ASTM C 494, annex A1. If mixers do not meet the test requirements, the batch size may be reduced or the mixing time increased. If they still fail to meet the requirements, they must be repaired or replaced.

b. Central mixers. - Mixers in centralized mixing plants shall be arranged so that the mixing action in the mixers can be observed by the inspector and plant operator from a location convenient to the mixing plant operator's station.

c. Truck mixers. - Truck mixers shall have an accurate water meter and revolution counter. They shall mix for between 50 and 100 revolutions at rated speed. On long hauls, there is a 1-1/2-hour time limit between the time the cement is introduced into the mixer and the concrete is discharged.

9. Temperature of Concrete (conc. 11). - Concrete canal linings placed during warm, dry weather shall not exceed 80 °F. Otherwise, the temperature of concrete when placed shall not be over 90 °F and not less than 50 °F. Concrete is best placed as cool as possible except during freezing weather. For mass concrete the maximum placing temperature is determined for each job depending on the need to control heat and temperature rise. For Crystal Dam, this was 50 °F and at Mountain Park it was 60 °F.

10. Forms, Finishes, and Tolerances (conc. 12, conc. 16, conc. 26). - There are four classes of finishes of formed surfaces: F1, F2, F3, and

F4.

An F1 finish applies to surfaces where roughness is not objectionable such as those upon which fill material or concrete is later to be placed.

An F2 finish is generally required on all permanently exposed surfaces such as canal structures and other concrete moderately exposed to view.

An F3 surface is required for structures prominently exposed to public view where appearance is especially important.

An F4 finish is required for formed concrete surfaces where accurate alinement and an even surface are essential to prevent cavitation.

Form material which we allow to be used depends on the type of finish which is required. For isntance, we allow the use of most rigid-forming materials, including any grade wood sheathing, when an F1 finish is specified; but, we require No. 1 common grade lumber plywood, or steel sheathing for forms to attain an F4 finish.

Finishes for unformed surfaces are designated by U1, U2, U3, and U4:

A U1 finish is a screeded finish that will eventually be concealed by fill material or concrete. Finishing usually consists of leveling horizontal surfaces with vibrators or screeding to produce uniform sloping surfaces.

A U2 finish is a "floated" finish applied to surfaces not permanently concealed and not requiring one of the smoother finishes. A continuation of the U1 finish, a U2 finish is obtained with a wood or magnesium float, either hand- or power-driven, to further smooth the surface. The floating should be as little as is needed to produce a surface that is free from screed marks and is uniform in texture. Finishers sometimes tend to overfloat and bring too much mortar to the surface at this state of the finishing.

A U3 finish is a continuation of the U2 finish. After a U2 finish has started to harden, troweling with steel trowels gives a smooth U3 finish. A U3 finish is generally applied to surfaces exposed to close public inspection.

A U4 finish is applied to canal and lateral linings being placed by lining machine. The surfaces are required to be equivalent in evenness, smoothness, and freedom from rock pockets and surface voids to that obtainable by effective use of a long-handled steel trowel. In other words, U4 is not required to be quite as smooth a finish as a U3 finish.

The tolerances for concrete construction are extremely important, but we suspect that past specifications have not been very helpful. Therefore, we have completely rewritten the tolerances paragraph keeping in mind that inspectors need to have specific, measurable requirements to enforce. The tolerances include surface irregularities and also deal with allowable deviations from specified locations and dimensions. The "tolerances" paragraph is no more effective than the inspector allows it to be. The inspector must use it in order for it to work.

10. Preparation for Placing (conc. 13). - Foundation surfaces on which concrete is to be placed shall be free from standing and running water, mud and debris, and free of frost and ice. Concrete surfaces against which concrete is to be placed and where bond or watertightness is desired shall be clean and dry. Cleaning of construction joints consists of removal of laitance, loose or defective concrete, wax-base curing compound, etc. Cleaning shall be done by wet sandblasting followed by washing with air-water jets or high-pressure water jets, except that a high-pressure water jet may be used in lieu of sandblasting on mass concrete. Construction specifications now require that placement checkout cards be used for concrete placements. Checkout cards are a tool used by contractor personnel and inspectors to assure that proper preparations are completed before concrete placement is begun.

11. Placing (conc. 14). -

a. The methods and equipment used for transporting concrete and the time elapsed during transportation shall be such that neither appreciable segregation of coarse aggregate nor a slump loss of more than 2 inches will occur in the concrete before it is placed. Aluminum pipe shall not be used for tremie, trunkline, or chute placing of concrete or for the delivery of pumped concrete as the resultant concrete will tend to expand. Equipment used must be able to place concrete of the specified slump. We do not allow the contractor to adjust the concrete slump or mix design to fit his inadequate or wornout equipment.

b. All concrete shall be consolidated by vibrators, either immersion-type or form vibrators. The depth of each layer of concrete which is individually consolidated must not exceed 20 inches. On steep slopes where internal vibration of an unconfined concrete surface would be impractical without forming, specifications are written to require use of a 30-inch-wide slip-form screed with concrete being placed ahead of the screed and vibrated by internal vibrators to get complete filling under the screeds.

c. Mixes for start of structural formed concrete. - Structural concrete placements should be started with an oversanded mix containing 3/4-inch maximum aggregate 2 to 6 inches deep at the construction joint.

We must strive to prevent poor consolidation at the construction joints and other areas particularly vulnerable to poor consolidation.

12. Repair of Concrete (conc. 15). - Repair of concrete shall be made in accordance with the "Standard Specifications for Repair of Concrete," dated January 4, 1982. As previously mentioned, this specifications was revised to reflect the advancements made in concrete repair and maintenance.

13. Protection (conc. 17). - The contractor is required to protect all concrete against injury until completion of the job. The fresh concrete must be protected against injury from rain and freezing weather. Fresh concrete surfaces should be covered in some manner to protect them against the "washing" effect of precipitation.

Concrete must be insulated and heated as necessary until it has gained adequate strength to prevent damage from freezing. Extent of protection against freezing is dependent on a number of variables such as curing and method of protection. All concrete must be maintained above 50 °F for 72 hours. Concrete cured with curing compound needs no further curing, but other concrete should be maintained at a temperature above freezing for another 72 hours to allow the concrete to dry somewhat before freezing. Where artificial heat is employed to maintain adequate curing temperature, specifal care must be taken to prevent the concrete from drying.

Concrete must also be protected from mechanical damage such as excessive foot traffic. Concrete in high traffic areas may be covered with plywood or other protective material to prevent excessive wear.

14. Curing (conc. 18). -

a. Water curing is still the preferred method where it is feasible. The water curing period is normally 14 days, during which time the concrete must be kept continually wet by water sprays and/or damp, absorptive mats. Water curing is normally required for horizontal construction joints in mass concrete dams.

b. Wax-base curing compound and recently, water emulsified resin base curing compound are usually permitted for canal and lateral linings and for structural concrete where water curing is not practical.

c. CRC-101 curing compound and recently, clear water emulsified resin base curing compound are used where architectural appearance is important. CRC-101 curing compound is harder to apply properly than either wax-base or water emulsified resin base compound. Improperly applied, this compound has devastated the appearance of architectural concrete.

d. Covering with polyethylene sheeting is an effective means of curing horizontal concrete members. The surface to be cured should be thoroughly moistened, the polyethylene sheeting should completely cover the surface, and it must be secured in a manner to effectively seal the surface and prevent air circulation.

Wax-base curing compound is also used as a means of preventing bond at a contraction joint. Although we require that all curing compound be removed from construction joints as part of the "pre-placement cleanup," some contraction joints are required to have a layer of curing compound to allow a functional crack to form at that point.

15. Reinforcement Bars and Fabric (conc. 25). - Except for minor uses, reinforcement bars are required to have the "high bond" type deformations. There are several types and grades, and we refer to ASTM specifications.

Before concrete is placed, the surfaces of the bars and bar supports shall be cleaned of heavy flake rust, loose mill-scale dirt, grease, or other foreign substances. Heavy, flaky rust that can be removed by firm rubbing with burlap is considered objectionable. Light rusting is not considered objectionable if it is firmly attached.

16. Measurement and Payment for Concrete (conc. 21 and 22). - In general, concrete is measured for payment as follows:

> When concrete is to be placed directly upon or against surfaces of excavation, measurement for payment is made to the same lines for which payment for excavation is made.
>
> For concrete not in contact with foundation surfaces, measurement for payment is made to the "neat lines" of the structures.
>
> Payment for concrete is made on the basis of the unit prices bid for the particular concrete. Payment for cement and for reinforcement is usually made separate from payment for concrete, but all other ingredients are included.

Prestressed Concrete

To date, use of precast, prestressed items consists mainly of beams, girders, roof tees, wall panels, and concrete pipe.

A. Prestressed Beams and Girders

Prestressed beams and girders have been used mainly in bridge construction. Prestressed tees are used in roofs of powerplants, pumping plants, control center buildings, and substation construction, and the panels have been used on walls of similar types of buildings.

On large powerplants and pumping plants, the contractor has had the option of either pretensioning or posttensioning of roof tees. More commonly, the stressing of the tees has been limited to the pretensioning method. We have also required the pretensioning method for bridge beams and girders.

Our bridge designers usually design the bridge beams and girders, and the construction specification are written to cover the materials and construction requirements. The specifications for the concrete are usually as follows:

1. The design concrete mix is usually based on a compressive strength of 5,000 lb/in^2 at 28 days. If 90 percent of the test cylinders exceed this, the mix is generally considered as meeting the accepted standards for prestressed concrete.

2. Three-quarters-inch or 1-inch-maximum size aggregate is used.

3. Cement content may not be less than six bags per cubic yard.

4. Entrained air is not required for freeze-thaw durability. Therefore, there is not a minimum limit on air content in precast concrete.

5. Use of admixtures containing $CaCl_2$ is not permitted.

B. Prestressed Tees

For roof tees, the Bureau has been using a product-type specifications. The specifications require the contractor to design the tees to withstand specific loadings including those induced during handling, shipping, lifting, and erection.

1. For large powerplants, the contractor has been permitted to factory cast or cast at the jobsite. More commonly, the tees have been required to be factory cast.

2. The manufacturer of the tees is required to have regularly manufactured precast, prestressed concrete tees for a period of not less than 5 years, and he must have facilities adequate to manufacture the tees of the size and type required.

3. The contractor is required to prepare shop and erection drawings and these are required to be submitted for approval before manufacture.

4. The design compressive strength is determined by the contractor's designers.

5. Sand and coarse aggregate shall conform to ASTM Designation: C 33, either 3/4-inch maximum size or 1-inch-maximum-size aggregate will be required unless otherwise approved.

6. Cement should be Types I, II, or III.

7. Water must meet standard requirements.

8. Use of admixtures containing $CaCl_2$ is not permitted.

C. Concrete Pipe

The Bureau uses several different kinds of precast concrete pipe, two of which are prestressed. The requirements for concrete in concrete pipe are very similar to the requirements for other precast concrete.

Bureau "Standard Specifications" for concrete pipe are available for reinforced concrete pressure pipe, unreinforced concrete drainage pipe, pretensioned concrete cylinder pipe, and embedded cylinder prestressed concrete pipe.

D. Curing

Precast concrete may be cured in four ways:

1. Water curing (low temperature)
2. Water curing (high temperature)
3. Steam curing
4. Radiant heat curing

Because low-temperature water curing is slower than curing at elevated temperatures, precasters almost never select water curing because it slows down their operation. Steam curing is accomplished by subjecting the concrete to enclosed steam heat; radiant heat curing is accomplished by covering exposed concrete surfaces to prevent heat and moisture loss and then circulating hot water or oil through pipes enclosed under the forms. High-temperature water curing involves dry heat in combination with water spray to keep the concrete wet.

The cures accomplished with steam, radiant heat, and high-temperature water cure are very similar, and the same precautions are necessary.

1. Do not heat the concrete up too soon (3 hours is a minimum).
2. Do not heat it up too fast (20 C°/hr is a maximum).
3. Do not let it dry during the curing period.

Water Systems Operation and Maintenance Workshop - 1990

CONCRETE REPAIR AND MAINTENANCE*

Session 9-2

Glenn Smoak, Dennis Arney, and Todd Rutenbeck

Introduction

Management of water systems is a field that is broad enough to stagger the imagination. Many disciplines in the technical and administrative communities are called upon to function either independently or jointly to effect the proper operation and maintenance of complex water works. Since most of the structures with which you are concerned are built at least in part with concrete, you are by necessity concerned and faced with concrete repair and maintenance work. The discussion contained herein is intended to assist you in developing good repair techniques which will result in serviceable and durable repairs.

The fact that repairs are required generally implies a failure in either materials or workmanship. Although you who are attending this workshop likely had no influence on the design or construction of the structures requiring repairs, you do have prime responsibility for assuring the continued safety, structural integrity, and operating efficiency of the aging structures under your jurisdiction.

Causes of Concrete Deterioration and Failure

Causes of concrete deterioration and failure can be grouped into three general classifications:

1. Planning, design, or construction deficiencies. - These causes of failure could be due to inadequate foundation explorations, structural shortcomings due to improper design, omissions in operating criteria, inadequate construction supervision resulting in flow surfaces outside of allowable grade tolerances, etc. Also, improper selection of construction materials can result in alkali-aggregate reactions, chemical attack from sulfates in soil or ground water, freeze-thaw damage, and structural failures.

2. Unforeseen conditions - Such unexpected conditions as floods, storms, earth and rock slides, lightning, etc., can cause minor to severe damage to concrete structures.

3. Natural attrition and old age. - All materials of construction are subject to aging and, concurrently, weathering and deterioration. The severity of the weathering process will depend upon the climatic conditions to which a structure is subjected as well as the type and quality of the original materials of which it was constructed.

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*This paper was authored by James R. Graham, formerly Chief, Concrete and Structural Branch; Todd Rutenbeck, Civil Engineer, Concrete and Structural Branch; and Orville R. Werner, Civil Engineer, formerly Concrete and Structural Branch, Division of Research, Engineering and Research Center.

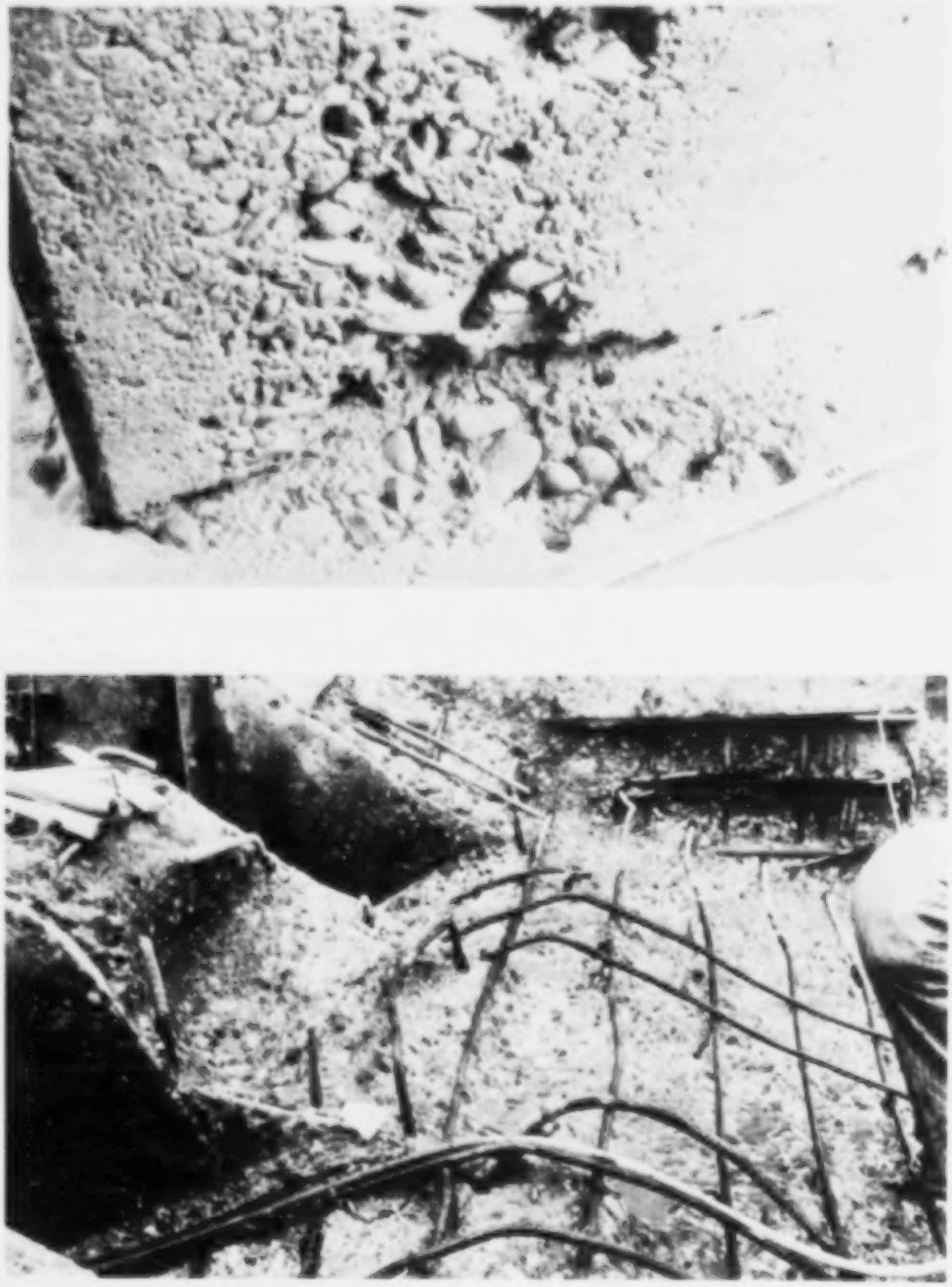


Figure 1. Two types of concrete damage caused by waterflow: cavitation damage in upper photograph; and abrasion damage from action of cobbles, etc., in water, lower photograph, Yellowtail Afterbay Dam, Montana. Photos C-8272-3NA and C-8272-23.

Maintenance and/or Repair

Generally, regular systematic maintenance will prolong the life of a structure and will often allow minor repairs to be made periodically, thereby precluding the need for major extensive repairs or replacement. Preventive maintenance could include such items as painting, providing better drainage, minor equipment replacements, etc. However, conditions may be such that minor or major repairs or replacements are required. In those cases, you would be required to make some decisions as to the extent of the failure and the methods of repair. Following is a brief outline which should help you in making those decisions.

Guide to Material and Repair Method Selection

When making a decision as to how to repair a structure, a logical systematic approach must be used. Following is a guide for making such a decision:

1. Determine the cause of failure such as chemical attack, foundation settlement, cavitation, abrasion, excess waterflows, materials failure, and freezing and thawing.

2. Determine the extent of failure by visual examination and/or use of some combination of nondestructive tests and coring.

3. Determine prerepair adjustments or modifications required, such as changing water drainage patterns, correcting differential foundation soils subsidence, correcting causes of cavitation damage.

4. Determine peripheral constraints such as availability of repair materials, access to the structure, operating schedule of structure, seasonal weather, and geographical location.

5. Determine the advantages and disadvantages of making permanent or temporary repairs.

6. Determine the repair methods available and technically feasible to use, such as concrete replacement, mortar replacement (shotcrete), dry pack, epoxy-bonded concrete, epoxy-bonded epoxy mortar, epoxy grout injection, or chemical grouting.

7. Determine from those repair methods previously termed technically feasible the particular one or combination of several that is most economically feasible.

8. Select the materials required to make the repairs based upon the particular technique chosen.

Standard Specifications for Repair of Concrete

The Bureau has a standard for repairing concrete entitled "Standard Specifications for Repair of Concrete, January 4, 1982." Although the specifications presently do not include details on all of the different repair methods, it does cover Bureau requirements for repairing spalled, defective, and damaged concrete. More information on these specifications can be obtained from the Bureau of Reclamation, Engineering and Research Center, attention code D-1510, Denver Federal Center, Denver, Colorado 80225.

Prerepair Preparations

Proper preparation of the surface of the existing concrete is one of the most important steps to making a good, serviceable repair. Since the bond of the new material to the existing concrete is just as critical to the success of the repair as is the quality of the new material, great care must go into this phase of the repair operation.

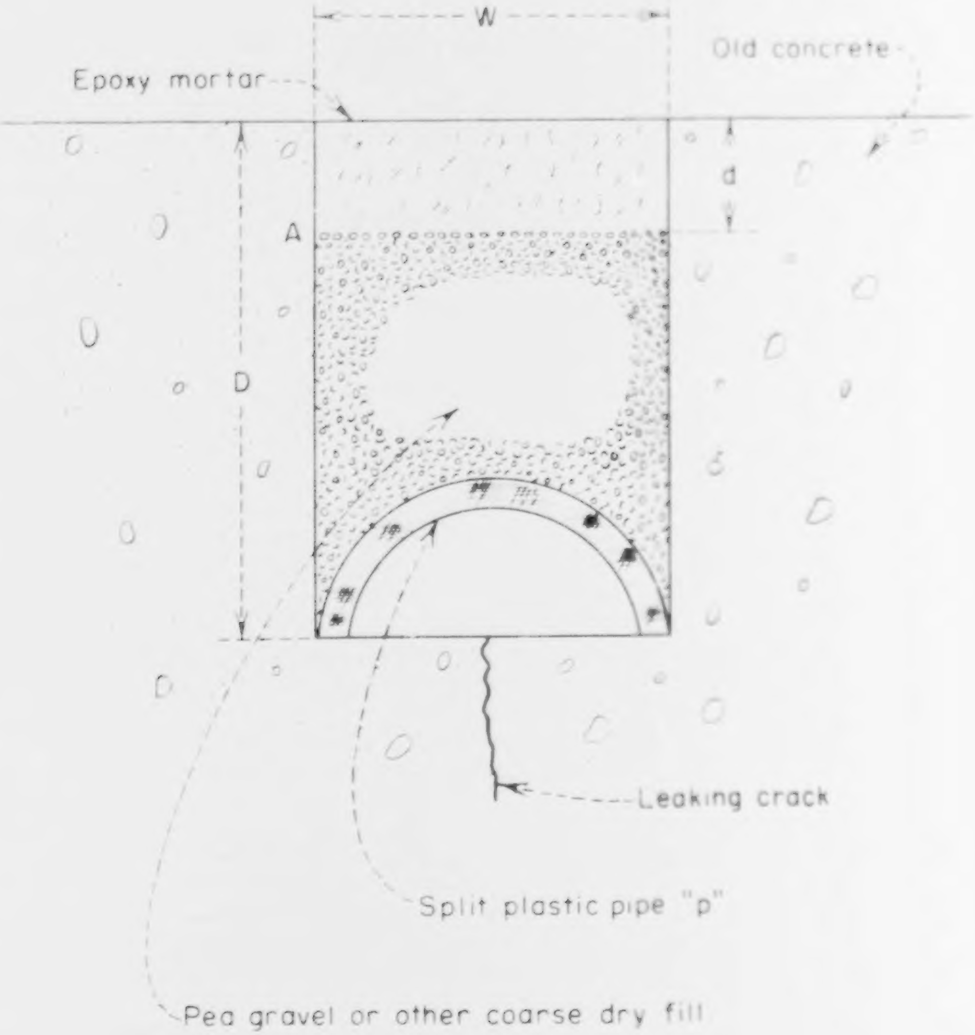
All defective or damaged concrete must be removed. No repair is better than the poorest material involved, particularly that material adjacent to the new-old material interface. If in doubt as to the actual condition of the existing concrete, remove another inch of material, especially in the case of sulfate or freezing deterioration. It is good practice to continue removing concrete until aggregate is being broken rather than loosened in the matrix.

In addition to the removal of defective concrete, surface preparation for new material is very important. The surfaces must be clean, rough, and dry when repair material is applied whether it be concrete, epoxy, or whatever. After the defective material has been removed, the surfaces must be prepared by wet sandblasting, water blasting with approved equipment, bush hammering, or any other acceptable method, and then cleaned and allowed to dry thoroughly. In the process of preparing the surfaces, care must be taken to prevent undercutting of aggregate in the existing concrete. It is advisable in the case of very old structures to wait several days after completing the surface preparation before applying the new material. Sometimes concrete in old structures seems sound when cleaned but will slake and soften after a few days' exposure. Care must be exercised to see that the cleaned and prepared concrete surfaces are maintained in that condition until repairs are made. Water seeps may be stopped by lead wool calking or by cover drains similar to those illustrated in figure 2. These may lead to sumps, to permanent weep holes, or to a vacuum system. Chemical grouting can also be used successfully, especially where leaks are from very fine cracks which are difficult to follow accurately with a routing tool, or in coarse, porous areas which are seeping. Any method of sealing which can be accomplished without contaminating the adjacent area is suitable for this work.

Discussion of Repair Techniques

The more common and widely used methods of repair such as concrete and mortar replacement, dry pack, and shotcrete will be discussed only briefly, thereby permitting a more extensive discussion of epoxy repair techniques.

Protective coatings. - Experience has shown that there are certain portions of exposed concrete structures which are more vulnerable than others to deterioration from weathering in freezing climates. The durability of such concrete features can be considerably improved and their serviceability greatly prolonged by preventive maintenance in the form of weatherproofing treatments. Selecting the most satisfactory protective treatment for concrete depends to a considerable extent upon correctly assessing the exposure environment.


W
Epoxy mortar
Old concrete
d
A
D
Leaking crack
Split plastic pipe "p"
Pea gravel or other coarse dry fill

d = 1/2" ±

W = To suit required drain size "p" should fit close.

D = Sufficient to provide drainage and permit drying of area "d" above "A".

CRACK AND JOINT DRAINS

Figure 2

Assurance that a situation requiring protection exists should be obtained before resorting to painting or a more frequent and severe maintenance problem may be created. Unless actual deterioration, such as that which painting will stop, is in progress, concrete usually will look better if left unpainted. It will also probably require less attention.

Some exterior latex and epoxy coatings are satisfactory for use in protecting concrete. Linseed oil coatings and linseed oil and pigmented paint have been used for many years as a protective material for concrete. Use of epoxy would involve a greater initial cost but would generally not require reapplication nearly as often as linseed oil.

Shotcrete repairs. - There are cases when it is feasible to repair deteriorated or damaged concrete by plastering with mortar. This can be done by hand, but more often than not, it is done pneumatically and is called shotcrete. Considerable skill is required by the operators to assure a quality repair since they have control of the amount of moisture in the placement and the application. It is quite difficult to get a pleasing finish using shotcrete; however, a satisfactory finish may be obtained by lightly rubbing the surface with a soft rag or burlap immediately after application.

Dry-pack repairs. - Dry-pack concrete patching is especially applicable where the repair area is generally small and relatively deep. (The depth must be nearly equal to or greater than the least dimension, and the sides relatively steep and confining.) This material, which must be rammed or tamped in place, results in patches, which when properly made and cured, are excellent in appearance and have excellent durability, often exceeding that of the basic concrete structure. A principal reason for using dry-pack patches is that such concrete has low shrinkage and less tendency to break its bond.

Preparations for and subsequent treatment and curing of dry-pack repairs should be similar to that for conventional replacement concrete repairs, as described below, except that curing compounds and excessive moisture should be avoided until the dry-pack has achieved some strength. Usually concrete repair is highly dependent upon its bond to the old concrete for success, and it should be especially noted that in dry-pack concrete repair, a bond coating must be provided in the area to be patched. This can be obtained by dusting the prewetted concrete with dry portland cement and subsequently brushing or blowing away the extra dry material. The bond coat can also be obtained by using a premixed mortar or grout. This is necessary because of the extreme dryness of the repair material, and no material can be expected to adhere unless it actually wets the original concrete surface. However, the bond coat must be very stiff; otherwise, it will tend to allow the dry-pack material to fall out.

Replacement concrete. - Repairs of conventionally cast concrete, like dry-pack repairs, require deep or vertical edges usually at least 1 inch in depth; and like all concrete repairs regardless of type, they require careful preparation and workmanship, and above all, extended moist curing. When the areas become larger and cast repairs are used, there will necessarily be higher water content and greater attendant shrinkage, and the actual bond strength of the patch to the old concrete becomes more difficult to obtain.

When bonding a conventional concrete patch, the old concrete must be "surface-dry." In this case, the preferred method of surface drying is by means of clean air jets, natural drying, or artificial heating. Good bond is obtainable whenever the surface is in a surface-dry condition. Free moisture on the surface should be avoided and can be detected by a glistening, shiny, wet appearance. The new concrete should be as stiff as is practical to place with available compaction methods but workable enough to assure actual moistening of the surface-dry concrete being repaired.

After placement, best bond strength will be developed when the patch is kept completely moist for at least 28 days. In some cases, curing by use of an approved curing compound or water spray followed by sealing with sheet polyethylene may be appropriate. It should be emphasized that in the matter of concrete repair there is no substitute for good workmanship; and especially where bonding of concrete is concerned, good workmanship includes the best of wet-curing. You can depend upon bond strength developing during normal curing of structural concrete where repairs are of considerable depth (8 inches or more). Tests in Europe and mathematical analyses have indicated that under normal circumstances drying is not likely to extend more than 5 or 6 inches into good concrete even over long periods of time; hence, bond strength will have adequate time to develop under moist conditions at a depth of 8 inches or more before significant stress is applied due to the drying shrinkage at the surface.

Figure 3 presents data originally reported by Messrs. Grieb and Werner 1/ of the Bureau of Public Roads. It shows that where concrete was wet-cured for 7 days only, the limit of tensile strength, which here is also considered to be a measure of bond strength, is somewhere in the neighborhood of 350 lb/in^2 regardless of the compressive strength developed by these concretes. In figure 3 it may be seen that, in all cases, concrete which developed a compressive strength of 3,000 lb/in^2 or greater also produced a tensile strength in excess of 350 lb/in^2 when wet- or moist-cured for 28 days. From this, it may be concluded that if conventional concrete repairs, either shotcrete, dry pack, or replacement concrete, are to be used, a minimum of 28 days' moist-curing should be provided.

Mr. Earl J. Felt of the Portland Cement Association, in a rather extensive investigation 2/ some years ago, examined many cases of bonded concrete repairs, flood and pavement toppings, and other concrete joints. He devised a shear bond test, of which we also use a modification, as a measure of the bonding stress of concrete. From this test, which invariably gives strengths higher than the indicated tensile strength of companion concrete specimens, he deduced that the shear bond strengths for durable repairs should never be less than 350 lb/in^2.

1/ "Comparison of Splitting Tensile Strength of Concrete with Flexural and Compressive Strength," Paper for the 65th annual meeting, ASTM, New York City, June 1962.

2/ "Resurfacing and Patching Concrete Pavement with Bonded Concrete," Proceedings of the Highway Research Board, vol. 35, p. 444, 1956.

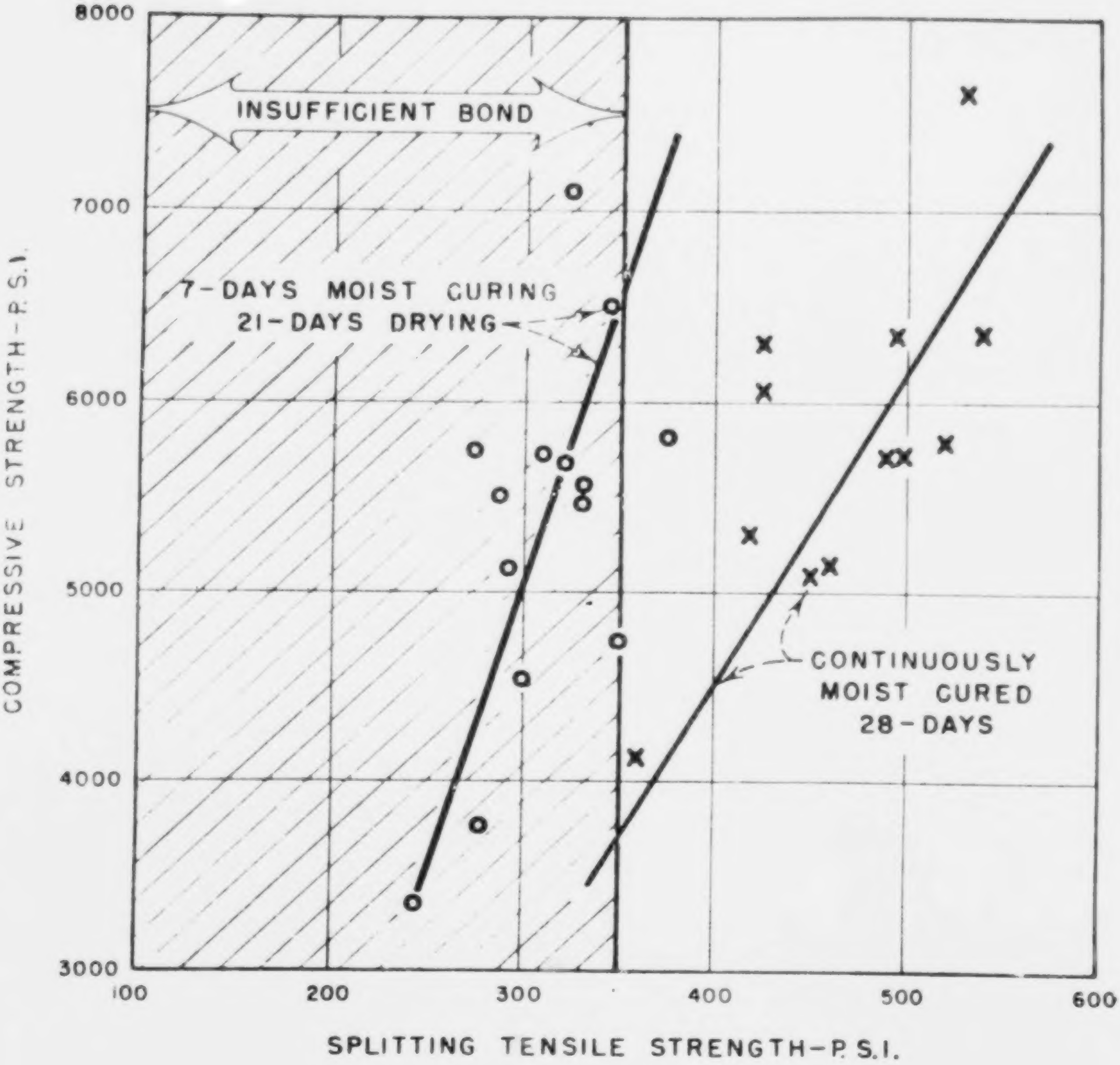


Figure 3

Modern epoxy adhesives may be obtained especially for the purpose of bonding concrete in structural work and may be used to bond a cast-in-place concrete repair to old eroded or damaged concrete where it is not feasible to provide continuous moist curing for at least 28 days.

In figure 4 are plotted curves showing the rates at which shear bond strength and compressive strength develop. The upper line is a typical curve for the development of compressive strength. The lower line is an actual measure of the shear bond strength of concrete, moist-cured to various ages up to 28 days. In each case, all values are represented as a percentage of the 28-day strength of that particular type, the 28-day strength being considered as 100 percent. As is apparent, the rates of strength development vary greatly. Shear bond strength after 7 days' curing is only 19 percent of the 28-day shear bond strength; whereas, compressive strength which is normally considered to be the criterion of structural concrete adequacy is somewhat greater than 60 percent.

Epoxy-bonded concrete. - When situations prohibit long wet-curing periods or when adequate wet-curing cannot be assured, chemical bonding agents may be used. Epoxy-bonding agents are the most widely used materials for such purposes.

Although epoxies and epoxy mortars are relatively expensive and hard to work with, they may be a logical choice because of their excellent resistance to freezing and thawing deterioration, because of the rapid completion of repairs that is possible, resulting in early use of a structure, and because concrete bonded with epoxy will develop adequate bond strength when cured in the same manner as normal structural concrete; that is, with membrane-curing compounds or by other covering and sealing methods.

Many proprietary epoxy formulations prepared for bonding old concrete to old concrete, new concrete to old concrete, and epoxy mortar to old concrete, are available on the market. Many of these materials are excellent high-quality products which can be used with a reasonable amount of certainty as to the results. However, some of the materials available are quite unsuitable for the majority of repair applications.

Epoxy bonding systems which meet the applicable requirements of ASTM C 881, "Standard Specification for Epoxy-Resin-Base Bonding Systems for Concrete," are generally suitable for bonding fresh concrete to hardened concrete. Moreover, a 100 percent solids system should be used. Although some epoxies which meet this specification are formulated for use at cold temperatures, best results will be obtained at moderate temperatures.

As previously mentioned, repairs which are at least 1-1/2 inches thick at minimum sections and not more than 6 or 8 inches thick usually require a bond coat be applied immediately ahead of the concrete by spreading activated liquid epoxy not thinner than 80 square feet per gallon on the old concrete. The area of coverage per gallon of agent depends on the roughness of the surface to be covered and may be considerably less than the maximum specified. The epoxy-bonding agent may be applied by any convenient, safe method such as squeegee, brushes, or rollers which will yield an effective coverage, figure 5. Spraying of low-viscosity systems is permitted if an efficient airless spray is used and when the concrete surfaces to receive the epoxy are 70 °F or warmer.

During the application of the epoxy-bonding agent, care must be taken to confine the material to the area being bonded to avoid contamination of adjacent surfaces. However, the epoxy bond coat should extend slightly beyond the edges of the

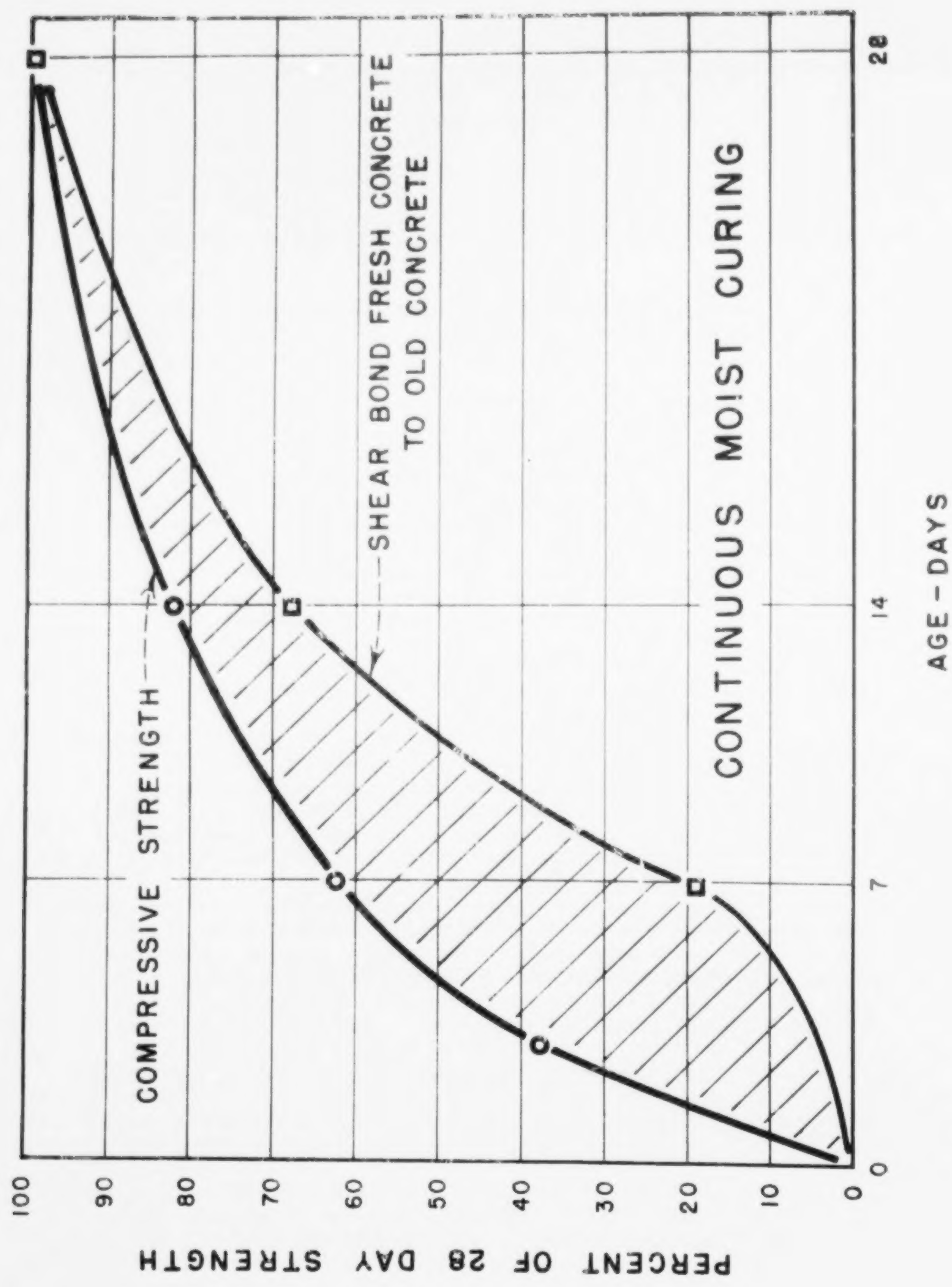

PERCENT OF 28 DAY STRENGTH
100
90
80
70
60
50
40
30
20
10
0
0
7
14
28
AGE - DAYS
COMPRESSIVE STRENGTH
SHEAR BOND FRESH CONCRETE TO OLD CONCRETE
CONTINUOUS MOIST CURING

Figure 4.

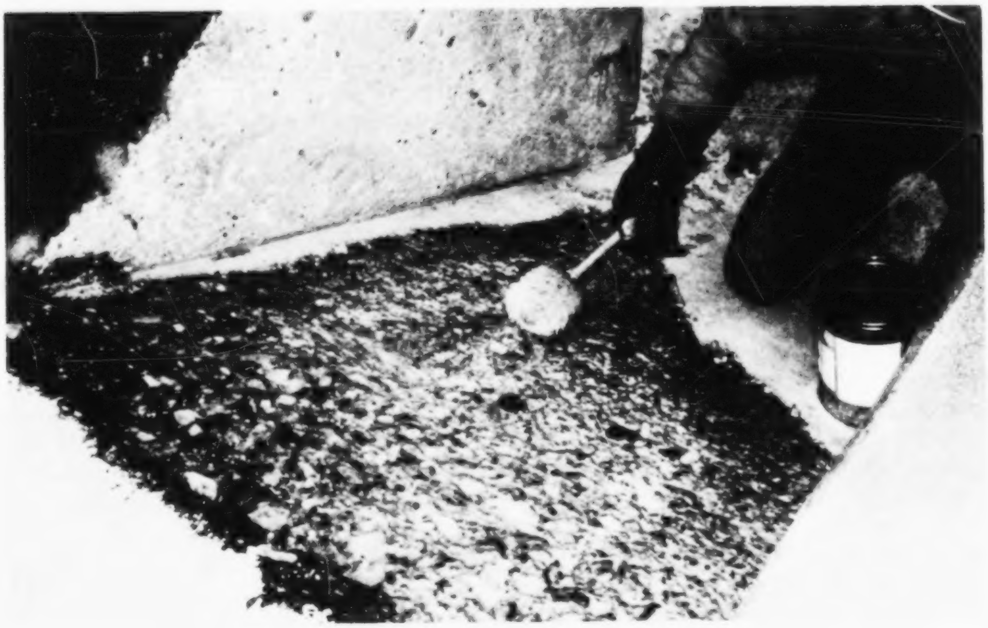


Figure 5. Application of epoxy bond coat to clean, dry, sandblasted areas adjacent to the dentates in the Yellowtail Afterbay Dam sluiceway. Photo C-8272-30NA.

repair area. If the repair work should be delayed, allowing the bond coat to stiffen so that the concrete mix does not penetrate into it, failure may result. Concrete placement must be accomplished while the bond coat is fluid, figure 6.

If concrete placement must be delayed, an additional bond coat should be applied while the first application is still tacky. Further, if epoxy hardens to a dry nontacky surface, it should be completely removed by sandblasting before applying a new bond coat.

An all-out effort should be made to secure a well-proportioned and well-mixed portland cement concrete having a 1-inch slump. The fresh concrete should be thoroughly compacted into the fluid epoxy bond coating. If concrete is placed immediately after mixing, set retarders or controlling agents should not be used as the epoxy may cure before the concrete sets.

Epoxy repairs to concrete, either bonded concrete or epoxy-bonded epoxy mortar, are entirely feasible in cool weather or under ground provided temperatures can be maintained above 40 °F for curing purposes. Epoxy cures adequately but slowly when maintained at 40 °F, although strength development is greatly retarded at cool temperatures. Concrete in tunnels or in contact with earth, especially below the water table, will be essentially the temperature of the earth or the ground water and be little influenced by air temperature. Frequently, such concrete temperatures are close to 40 °F although air temperature is maintained at 70 to 80 °F for a long period of time. Preferably, temperatures for good workmanship with epoxy mortar and epoxy-bonded concrete should be at least 60 °F. Epoxy not only sets very slowly at lower temperatures, but is very difficult to work due to the increased viscosity; and workmanship may suffer with resultant imperfect bonding. It is necessary to obtain complete wetting by the epoxy which, when cold, spreads much like cold molasses on frozen bread. Warm temperatures are also desirable for the first 4 hours of hardening. After the first 4 hours, temperatures may be raised by radiant heat to accelerate final hardening, or may be allowed to go lower, providing temperatures above 40 °F are maintained until hardening is complete. Care must be taken when raising the curing temperatures to prevent the concrete from drying out.

When the concrete is too cold for effective use of epoxy systems, warming can be accomplished even on concrete in contact with the earth by use of bottle-gas-burning weed burners, or radiant heaters, figure 7. Space heaters are less effective because high air temperatures must be maintained for a long period when concrete is in contact with earth. Areas such as dentates above ground could well be enclosed in tents and the air heated until concrete reaches suitable working temperatures. Floor areas of walls in contact with the earth can be heated by gas-burning weed burners by moving the burner back and forth over a relatively small area until a hand can no longer be held on the surface. Radiant heaters, either gas or electric, are very useful for preheating cold concrete in these situations. Such heat should be applied for several hours in order to provide in depth heating. Concrete should then be allowed to cool until it feels only slightly warm or neutral to the hand. This provides some in depth warming and prevents sudden chilling of the epoxy. The setting process of epoxy is accelerated by higher temperatures and inhibited by lower temperatures. Heat is evolved during setting and heat evolution is accelerated at high temperatures. In fact, temperatures above 100 or 110 °F during early stages of epoxy curing may cause a runaway reaction, producing excessive heat and unacceptably high temperatures where thermocontraction will later cause failure. However, final curing of epoxy after initial hardening may be

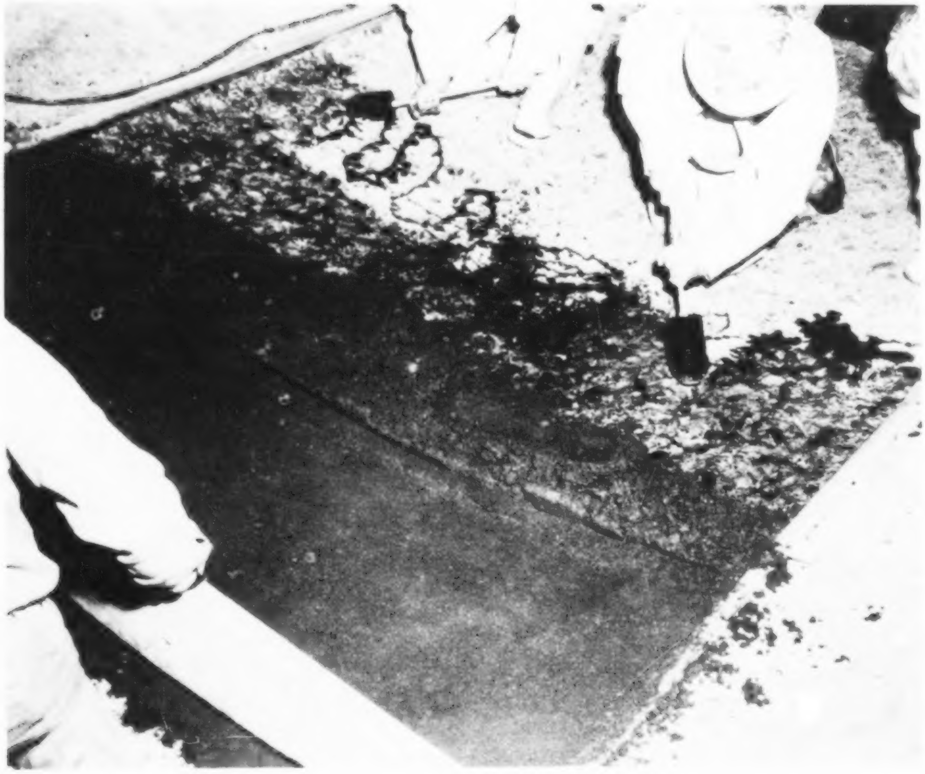


Figure 6. Application of epoxy bond coat on clean, dry, warm concrete surfaces immediately followed by placement of well designed low slump concrete. Photo P-622A-427-10535 NA.

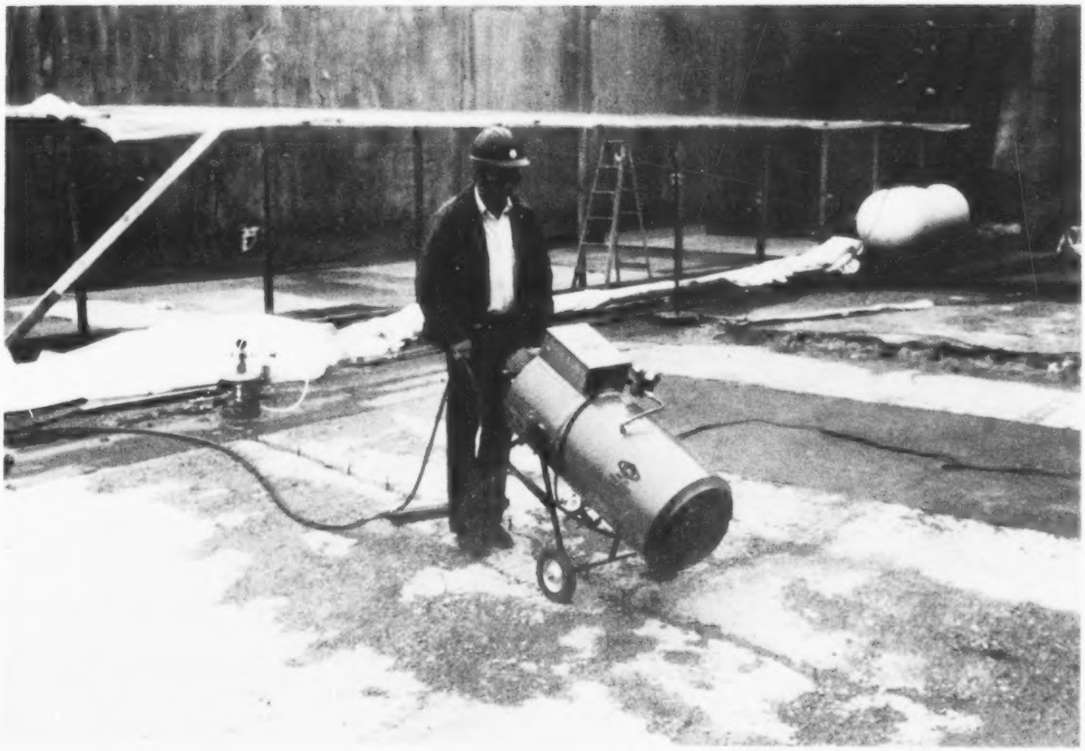


Figure 7. Warming and drying the previously sandblasted surface of concrete in preparation for application of epoxy-bonded epoxy mortar (Shadow Mountain Dam).

accomplished by applying temperatures of 90 to 110 °F for 3 or 4 hours. Care should be exercised to provide temperatures in the acceptable curing ranges at least during the early period of epoxy cure and not below these ranges during the entire time until final curing takes place.

Epoxy-bonded Epoxy Mortar Repairs

Epoxy adhesives called binders may also be used to produce complete repairs without use of portland cement or conventional concrete. Mortars may be manufactured from the binders and clean, dry sand. These mortars are particularly useful in patching spalls or cracks. Such repairs may be featheredged and cured quickly, especially when applied to warm surfaces and/or heated after application. Numerous kinds of repairs are possible with epoxy mortars. For instance, they are frequently used around the periphery of epoxy-bonded concrete to permit featheredging, instead of requiring additional excavation of old concrete.

Caution should be exercised when epoxy-bonded concrete or epoxy-bonded epoxy mortar repairs are made in spalled areas adjacent to joints or along cracks which may be subject to future movement. It should be remembered that good workmanship will produce repairs which are harder and stronger than the original concrete, and future movement, if it does occur, would probably produce another crack adjacent to the repair. When such action could produce a serious maintenance problem, consideration should be given to some type of flexible repair which can accommodate the movement.

Epoxy mortar is prepared by mixing a known quantity of well-graded, clean, dry sand with a known quantity of activated epoxy. The epoxy should meet the applicable requirements of ASTM C 881, with the following additional requirements: (a) It should be a 100 percent solids system, and (b) It should meet the strength requirements for a Type II system. The grading and condition of the sand have a very marked effect on the workability and quality of the epoxy mortar. Therefore, the sand must be of good quality, clean, and should conform within reasonable limits to the gradings in the following chart:

Aggregate Grading for Maximum
Efficiency in Epoxy Mortar

Maximum size	Grade percent retained							
	3/8 inch	#4	#8	#16	#30	#50	#100	Pan
#30						50	30	20
#16					31	23	16	30
# 8				31	22	16	10	21
# 4			19	21	15	10	8	17
3/8 inch		29	21	16	10	7	4	13

Oftentimes, the quantity of pan-size material required is not available in a given sand and some has to be added from another source, such as from a manufactured supply. If the size of the pan material is such that most of it will pass a No. 200 screen, then the quantities of pan shown in the chart should be reduced by about one-half.

When efficient grading of sand for epoxy mortar is not practical due to the extremely small amount being required or the inaccessibility of equipment for providing such grading, any clean, thoroughly dry sand which does not contain excessively large sizes may be used. Reduction of maximum size to one acceptable for the application may be accomplished by screening through one or more layers of ordinary window screen. Several layers folded closely together can produce results nearly equivalent to the use of a very fine screen. Gap-graded or one size sand, such as that prepared for sandblasting, is not generally satisfactory for use in making epoxy mortar.

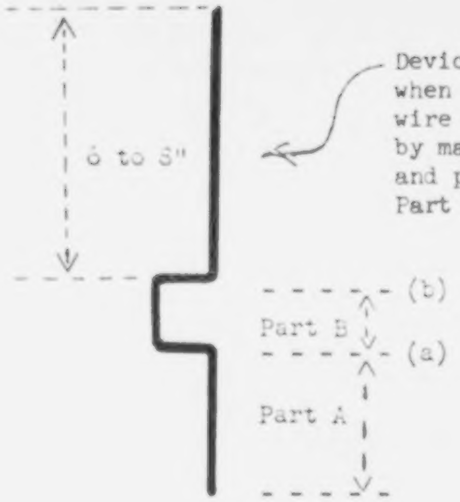
In most cases, when a mortar is prepared using a sand conforming to the aforementioned grading and an appropriate epoxy, a ratio of 5-1/2 parts graded sand to 1 part epoxy by weight provides a high-quality mortar. Such a mortar generally will provide a dense repair capable of being tooled to grade. Decreasing the sand-to-epoxy ratio will increase the density or impermeability of the mortar but will make it more difficult to place.

Epoxy has a tendency to promote thermal incompatibility with other materials, especially when the temperatures range from those of summer to well below zero. A lean mortar tends to have lower coefficient of thermal expansion, closer to that of other materials, and is less subject to thermal degradation. If finishing is a serious problem due to stickiness, some improvement may be had by using a coarser grading of sand. One should remember, however, that this tends to produce a porous mortar. If a slick finish or impermeability is needed, richer mortars with additional fines are necessary. In some cases, glaze coats of unfilled epoxy may be added after initial stiffening of the mortar. The addition of larger maximum size or coarser grading will usually increase the strength of the resulting mixture provided grading is continuous and uniform. Even poorly graded porous mortar is usually stronger in compression than common structural concrete. Maximum sizes of sand used will depend upon the particular application. Sand should be of the largest maximum size that can be properly finished or featheredged, if the application involves featheredging.

Two simple devices that have been useful in proportioning and mixing small batches of mortar are shown in figure 8. Larger batches, in some cases, have been successfully mixed in mortar mixers similar to those used by the plastering trade.

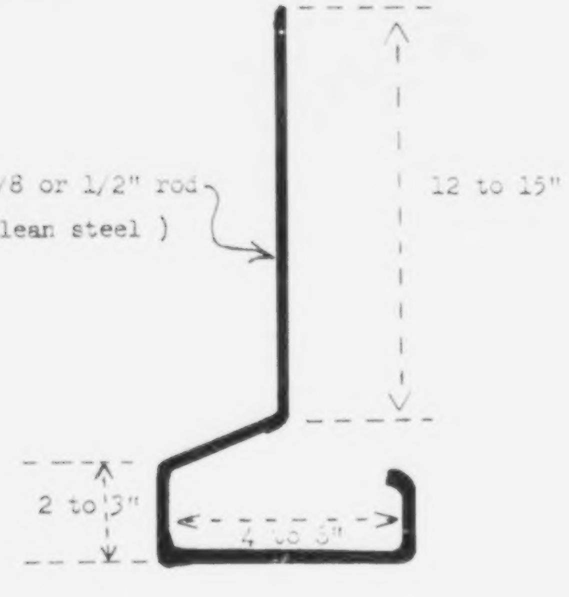
The prepared epoxy mortar should be tamped, flattened, and smoothed into place in all areas, while the epoxy bonding agent is still in a fluid condition. In the case of application on steep slopes or overhead, the epoxy bond coat may be allowed to become tacky before the mortar is placed. No epoxy mortar is to be placed in areas without the epoxy bond coat having been applied. The epoxy mortar should be finished with a steel trowel in slow, even strokes, figure 9. No thinners, diluents, water, or lubricants can be used on the trowels when finishing the mortar. However, warming the trowel will enhance the trowelability of the mortar.

Epoxy mortar applied on steep, vertical, or overhead areas has a tendency to slump out of place before final set. This may be alleviated to some extent by adding to the mix extra material in the very fine gradings, such as a small amount of very fine silica material. These materials stiffen the epoxy resin to reduce the tendency for sagging. Also, Grade 3 epoxy (non-sag) can be purchased with the fines already added. Repairs of such areas are also


6 to 8"
(b)
Part B
(a)
Part A

Device for volumetric proportioning of epoxy when small batches are desirable. Bend stiff wire as shown with proportion as recommended by manufacturer. Hold device in mixing container and pour Part A to first bend (a); then pour Part B to upper bend (b).

(use clean unpainted wire)


3/8 or 1/2" rod
(clean steel)
12 to 15"
2 to 3"
4 to 6"

Mixing device for epoxy and epoxy mortar to be used with slow speed heavy duty drill, preferably about 500 rpm or slower.

Figure 8

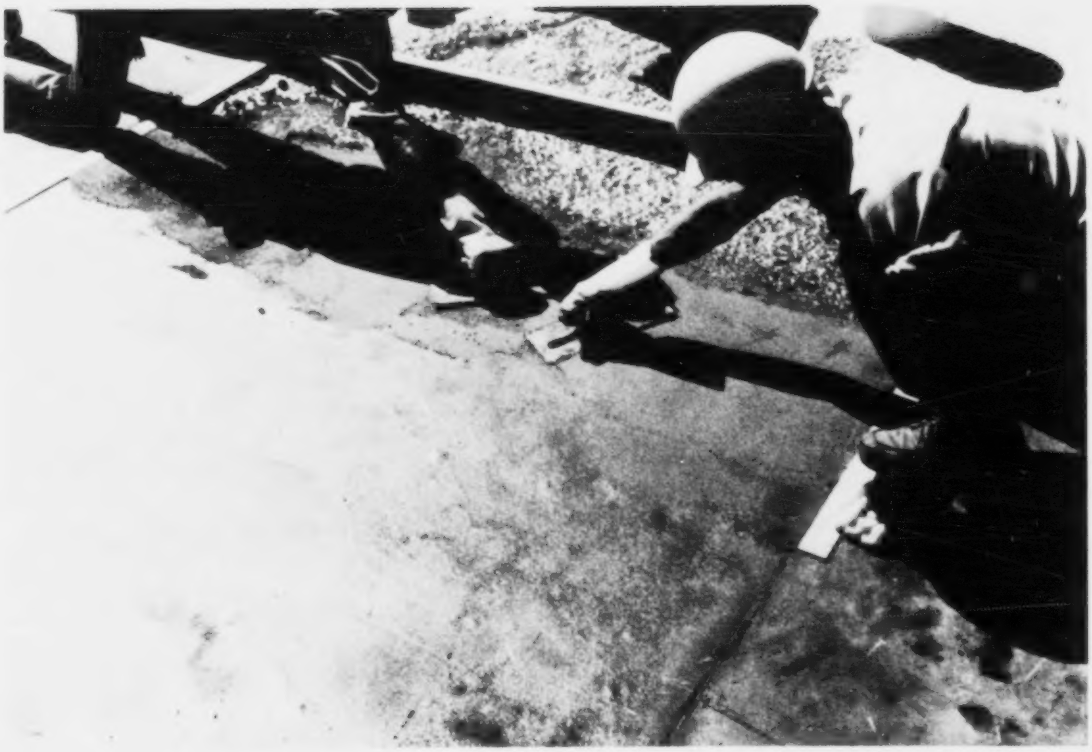


Figure 9. Epoxy mortar has been screeded and is being finished by short, even strokes of the steel trowels. Note the wood strips used as screed rails. These strips are needed to elevate the screed since epoxy mortar is "fluffy" and is somewhat compacted during finishing (Shadow Mountain Dam).

frequently accomplished by applying epoxy mortar to the bond coat in very shallow layers after the bond coat has become quite sticky or tacky. The layer applications are then repeated while the previous layer is very stiff but still slightly tacky like scotch tape. Forms covered with polyethylene sheeting or cellophane may be used to support such layers on the final grade. Small areas may sometimes be supported by simply spreading a sheet of polyethylene over the repair area and patting to flatten with a float or straight edge. Never attempt to remove such covering until after final hardening of the epoxy.

A slight running or sagging, or a featheredge which is not smooth enough, may be corrected by grinding after epoxy is thoroughly hardened. Such grinding is usually more difficult than grinding of concrete.

Epoxy mortar should be cured immediately after completion of each repair area at about 40 °F until the mortar has reached sufficient strength. When the mortar is hard, postcuring at 90 to 110 °F to heat in depth the epoxy mortar and subbase beneath the repair will optimize the quality of epoxy mortar and accelerate the curing process. Postcuring should continue for about 4 hours. Such heating can be accomplished by using portable propane-fired heaters, batteries of infrared lamp heaters, or any other acceptable method. The mortar must not be subjected to moisture until after the specified postcuring has been completed.

Epoxy pressure-injection repairs. - An effective method for structurally repairing cracks in structural members such as walls, piers, floors, ceilings, pipe, etc., is by injecting activated epoxy into the cracked area using pressure, figure 10. If cracks to be injected are clean and dry and the epoxy properly mixed, placed, and cured, the repaired member will have been restored to its original structural integrity. Although damp or wet cracks can be repaired using this method, development of sufficient bond between epoxy and concrete is much more difficult.

The epoxy should meet the requirements of ASTM Specification C 881 for a Type I, Grade 1 bonding system, and shall be formulated to be applied in moist environments, as applicable.

Handling of Epoxies

Certain safety precautions are necessary when handling or using epoxy resin materials. Epoxies and especially their activators are chemical irritants which can cause severe skin irritations or other disorders. Protective creams, gloves, goggles, and clothing which are disposable or can be laundered before reuse should be provided in all cases. Some sources of safety equipment and materials for handling epoxies are given in the appendix.

Every care should be taken to avoid contact of epoxy compounds with human skin; if skin contact does occur, the contamination should be immediately wiped off with a disposable wipe (paper towel) followed by immediate, thorough washing with soap and water. All personnel should be alerted to the potential hazard of using epoxies and carefully instructed to take all precautions necessary to avoid contamination which could result in discomfort or injury to the individual. Also, compliance with the OSHA (Occupational Safety and Health Act) Standards, Part 1910, is very important.

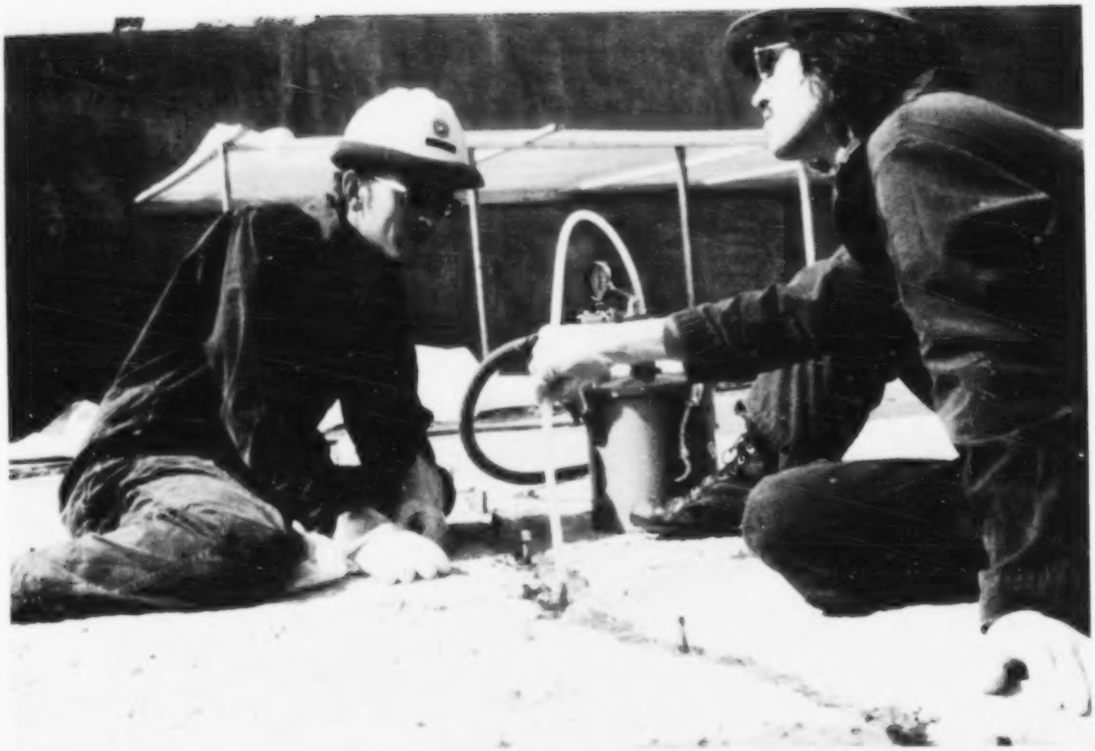


Figure 10. Epoxy resin is injected pneumatically into a crack in a slab at Shadow Mountain Dam. Note the use of protective clothing to protect against injury in the event of equipment failure under pressure.

Other Repair Materials

The repair methods previously discussed are generally applicable to relatively small areas. As you know, there are cases where large areas, such as roadways, etc. must be repaired. Petroleum products, such as asphalt, are available and have been used. Also, many synthetic materials, such as latex, acrylic, and polymer systems, are available. Neither the scope nor the capacity of this discussion permits a full treatment of the many materials that are available. Therefore, no further discussion of these products will be made.

Conclusions

Every facet or phase of concrete repair is important and critical to the success of the final product. Workmanship, material selection and treatment, and maintenance are all areas that if properly considered will ensure that a serviceable, long-life repair has been made.

Included in the appendix are the following items which should be of use to you:

1. Sources of safety equipment for use during epoxy application.
2. Supplies which may be necessary for concrete repair work.

For general reference, further details concerning the repair of concrete are contained in the Bureau's "Standard Specifications for Repair of Concrete," dated April 1, 1981.

In conclusion, and to verify some of the statements made herein through the writings of an old water works operator, the following is quoted:

> "Requirements for Good Concrete - Repairs that can be carried on without cutting off the water of the aqueducts consist principally of masonry work, which should be executed at the right time, and conscientiously. The proper time for masonry work is from the first of April to the first of November; but with this restriction: That the work be interrupted during the hottest part of the summer; because moderate weather is necessary for the masonry properly to absorb the mortar and to solidify into one compact mass; for the heat of the sun is no less destructive to masonry than is too violent frost. Nor is greater care required upon any works than upon such as are to withstand the action of water; for this reason, all parts of the work need to be done exactly according to the rules of art, which all the workmen know, but few observe."
>
> Excerpted from the Water Supply of the City of Rome of Sextus Julius Frontinus, Water Commissioner of the City of Rome, A.D. 97.

CONCRETE REPAIR

APPENDIX

Sources of Safety Equipment for Use During Epoxy Applications

Protective Creams

Ply No. 9	Milburn Company Detroit MI
Kerodex No. 71	Ayerst Laboratories, Incorporated 20 East 40th Street New York NY 10016

Gloves (1-3/4 mil polyethylene)

Wil - Gard	The Wilson Rubber Company Canton OH
Hand - Gard	The Carl H. Biggs Company 1547 - 14th Street Santa Monica CA 90404
Cyralon Industrial (Reusable) Estimate $16 per dozen	Skyline Manufacturing Company P O Box 185 Van Nuys CA 91408

Supplies Which May Be Necessary For Concrete Repair Work

1. Epoxy meeting the applicable provisions of ASTM C 881.
2. Safety equipment
   - a. Supply of polyethylene or rubber gloves.
   - b. Goggles.
   - c. Good supply of paper towels and disposal container.
   - d. Bar of soap for hand washing (not a detergent).
   - e. Bucket or tub of wash water.
   - f. Protective cream (Kerodex 71 or Ply No. 9, or similar).
   - g. Complete rain suits (rubber or plastic) for protection of epoxy handlers on extensive jobs.
   - h. Respirators (if work is in a confined or poorly ventilated area). These should be of Mine Safety and Health Administration-approved type for organic vapors. American Optical Company R-5051P with R-51 cartridges are suitable. For dust protection, use R-49 filters with these and Norton 7500-30 masks.

3. Slow-speed mixing equipment (large air or electric drill with stirrer or simple iron rod hook) not over 1,000 r/min; 500 r/min or less is better.

4. Very stiff brushes (large supply, small to medium size) for spreading epoxy bond coat. Useful life will be 1 hour or less for each brush. In some cases of special surface conditions, rags may suffice for this purpose.

5. Trowels, putty knives, survey stakes, etc., for placing and finishing.

6. Disposable buckets or other containers for mixing epoxy and epoxy mortar (1- to 3-gallon size polyethylene or 5-gallon fiber containers are suitable; should be tight and tough). Plan to need several.

7. Polyethylene sheeting for protection of work areas and/or for lining forms.

8. Sandblast equipment for cleanup.

9. Surface grinder and coarse abrasive disks (auto body shop type) for final finish on featheredge.

10. Solvent for tool cleanup only.

11. Weed burner, LP gas type for warming and drying repair surface. Heat lamps should be used if work is in tunnel. Manchester "Power Jet" LP gas hand-burner by Manchester Tank and Equipment Company, 2880 Norton Avenue, Lynwood, California, is an excellent burner.

12. Steel plates (thickness not critical) which can be supported near repaired surface to distribute heat from a torch or other spot source for accelerated curing of epoxy coating or epoxy mortar.

13. Plastic pipe if repair area is influenced by leaking holes or cracks. Diameter should be such that split piece will fit in a groove cut by airhammer or drill.

14. Airhammer or drill to remove weakened concrete and to cut drain channels if needed.

15. Adequate dependable pumps, preferably self-priming.

16. Clean, dry, well-graded sand for epoxy mortar. Maximum size depending upon size of repairs and degree of featheredging required.

17. Facilities for batching, mixing, and placing very stiff (1-inch slump) high-quality concrete if repairs are more than 2 inches deep and extensive.

18. Warm storage for supplies (70 to 75 °F).

19. Provision for forming or otherwise supporting repairs on steep slopes or overhead.

Water Systems Operation and Maintenance Workshop - 1990

EARTH CONSTRUCTION PRACTICES*

Session 10-1

Bob Baumgarten and Jeff Farrar

Foundation Considerations

In any study of a foundation, the first objective is to discriminate between sound and unsound foundations, and in practice to classify a foundation as adequate, inadequate, or questionable. At first, most cases will seem to fall in the questionable category but with increased experience these will decrease. To supplement judgment, test procedures have been developed for evaluating foundation and conditions. These may vary from simple index tests and visual observations to more elaborate sampling and laboratory and field tests.

Exploration Requirements

In the exploration of any foundation, it is of little value to examine only the exposed surface because the subsoil strength at depth determines footing requirements. There are rules for reasonable depths of investigations based on the theoretical distribution and extent of influence of pressures under a foundation. For footing foundations, these are shown in figure 1 and for embankments and cuts in figure 2, which are figures 39 and 41 extracted from the Bureau of Reclamation Earth Manual [1]**, pages 102 and 104.

In a foundation, we are interested in knowing the condition of the soil. It is, of course, of value to know the type of soil such as clay, silt, sand, or gravel, which could be determined by sampling from boreholes. But unless the samples permit us to interpret the firmness and denseness of the in-place soil, we cannot fully determine the foundation supporting capacity. We have a way of investigating a foundation through the determination of driving resistance using a standard drive point and a standard driving hammer. This penetration resistance test is shown in figure 3. (The procedure is described in test designation E-21 of the Earth Manual, page 603.) By counting the number of blows per foot of drive, this test permits us to evaluate the firmness of clays and the denseness of sand using information shown in figure 4 (see Earth Manual, pages 296-298). Also, this test has been found useful to help predict the driving resistance of piles (Earth Manual, page 302). When the foundation problem is more complex, detailed investigations are required. The Earth Manual test designation E-2 (page 341) describes several ways of obtaining undisturbed samples for laboratory testing. In problems concerning shear of saturated soft clays, we have the in-place vane shear test shown in figure 5. The basis of the test is to determine the torque (rotary force) required to shear the soil by rotating the vanes. The procedure is described in test designation E-20 of the Earth Manual, page 593.

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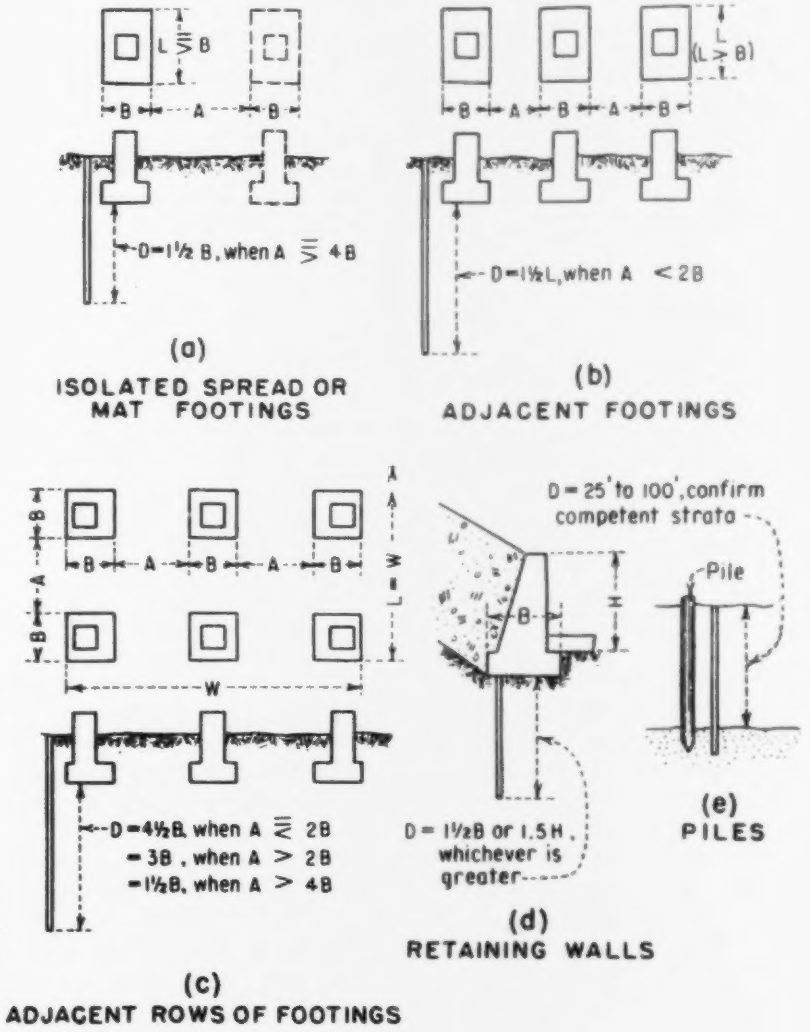
*For definitions of some common soils engineering terms, see the appendix.

**The numbers in brackets refer to items in the list of references at the end of Session 10-1.

The extent of testing depends on the importance of the structure, the seriousness of the soil conditions, and the steps that may be necessary to solve the problem.


L ⋝ B
B
A
B
D=1½ B, when A ⋝ 4B
L
(L > B)
B
A
B
A
B
D=1½L, when A < 2B
B
A
B
B
A
B
A
B
L = W
W
D=4½B, when A ⋜ 2B
=3B, when A > 2B
=1½B, when A > 4B
D=25' to 100', confirm competent strata
B
H
D = 1½B or 1.5H, whichever is greater
Pile

(a)
ISOLATED SPREAD OR MAT FOOTINGS

(b)
ADJACENT FOOTINGS

(c)
ADJACENT ROWS OF FOOTINGS

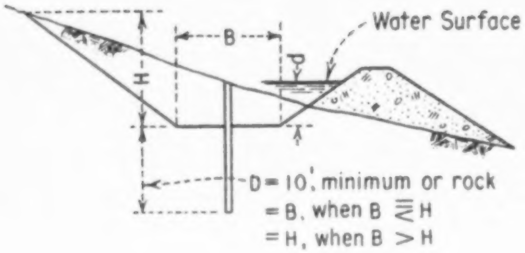
(d)
RETAINING WALLS

(e)
PILES

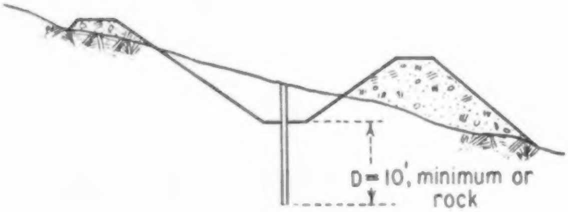
Figure 1. - Depth of preliminary exploration holes for point structures.

Bearing Capacity

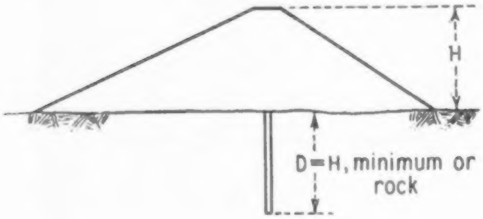
The foundation must be safe against punching into the ground. This may involve both shearing properties and consolidation properties. A saturated weak clay can be visualized as resulting in settlement of the soil under the foundation accompanied by bulging of the soil out from



(a)
DEEP CUT AND FILL SECTIONS



(b)
NORMAL CANAL SECTIONS



(c)
HIGH EMBANKMENTS

Figure 2. - Suggested minimum depth of preliminary exploration holes for canal and road alinement.

around the foundation. A large, rigid structure sometimes causes catastrophic failures because of this shearing weakness; however, when a soil is found to be firm, it can resist this weakness. Penetration resistance, vane tests, density tests, and laboratory testing of undisturbed samples are ways of evaluating this firmness. Improving the foundation with regard to bearing capacity problems can be accomplished by (1) deeper foundations, (2) overexcavation and refill, (3) piles to firm material, and (4) larger foundation areas to reduce load.

Stability

Stability of cuts and slopes is dependent largely upon the shearing characteristic. For example, a sand without cohesion will not stand in a vertical trench, yet a clay stands in a vertical trench up to a

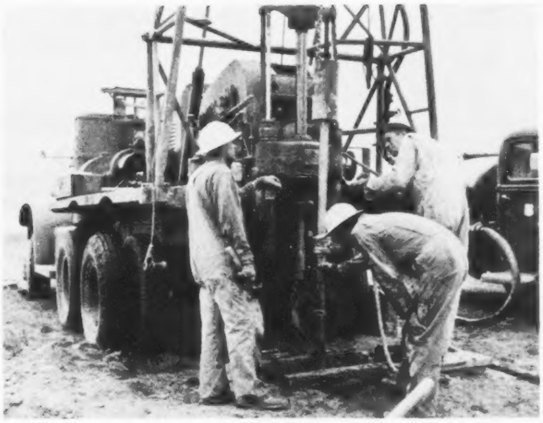


Figure 3. - Field penetration test equipment used by Bureau of Reclamation.

certain height depending on its amount of cohesion. This is called the critical vertical height and it would certainly be dangerous to dig a trench to depths near that of critical height. Current OSHA (Occupational Safety and Health Act) standards must be adhered to on all excavation work. The Earth Manual (page 196) gives a lengthy discussion on slope excavations, indicating that permanent slopes are commonly excavated at 1-1/2:1 and 2:1 and temporary slopes at 1:1. However, slopes in dry loessial soil where the binding strength is high are sometimes excavated at 1/4:1 or 1/2:1, as shown in figure 6. If these slopes become wetted and the binding strength decreases, it is obvious that the critical height will become lower and failure will result. Some clay soils soften and swell when wetted, and weaken to the extent that much flatter slopes fail, as shown in figure 7. Canal side slopes of marginal stability should never be subjected to rapid drawdown during unwatering operations as side slope sloughing is likely. A rule-of-thumb procedure when unwatering canals is to lower the water level not more than a foot a day to permit the soil-water pressures within the soil to dissipate.

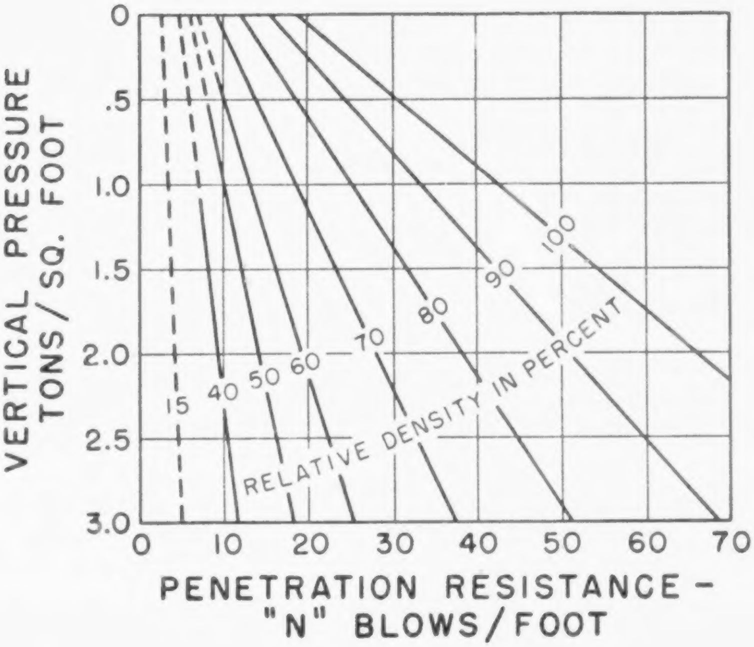


Figure 4. - Chart for estimating the firmness of clay and the denseness of sand based on the field penetration test.

An important and simple rule to follow is to observe the success of slopes already existing in the area for indication of expected stability.

Generally, slopes can be stabilized by (1) flattening the slope, (2) weighting the toe of the slope, (3) unloading the top of the slope, (4) dewatering the zone of slippage by draining, or (5) stabilization of clay by the addition of lime.

Settlement and Uplift

Some settlement can always be expected for foundations resting on soil. However, it is important to keep these settlements within tolerable limits. When "no settlement" is permitted, the foundation needs to be placed on solid rock or on piers, piles, or caissons to solid rock. Some settlement is not objectionable if it is not of appreciable differential amounts beneath the structure. The Leaning Tower of Pisa, figure 8, is a classic example of differential settlement. In hydraulic

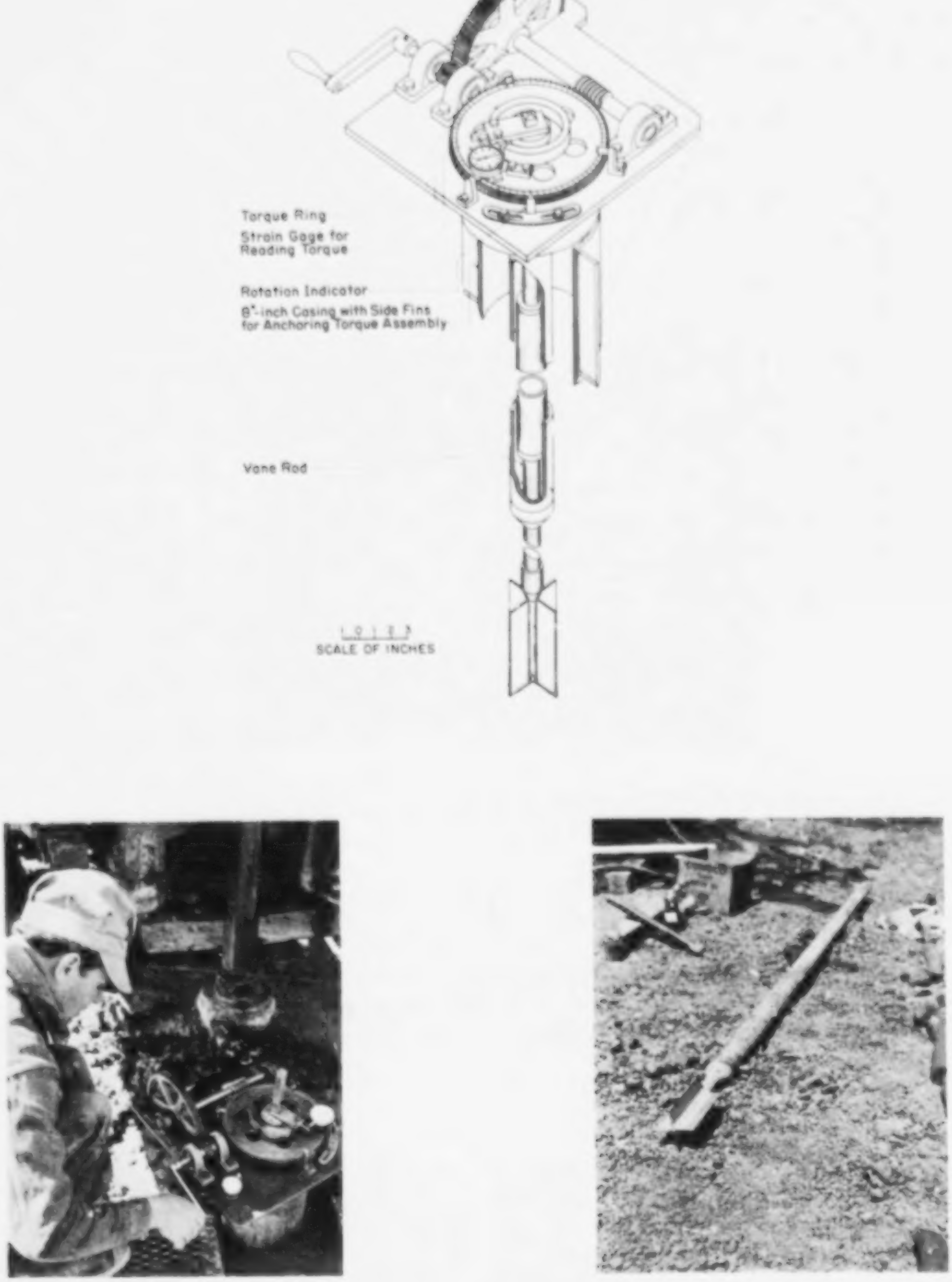

Torque Ring
Strain Gage for Reading Torque
Rotation Indicator
8"-inch Casing with Side Fins for Anchoring Torque Assembly
Vane Rod
1 0 1 2 3
SCALE OF INCHES

Figure 5. - Bureau of Reclamation vane test apparatus.

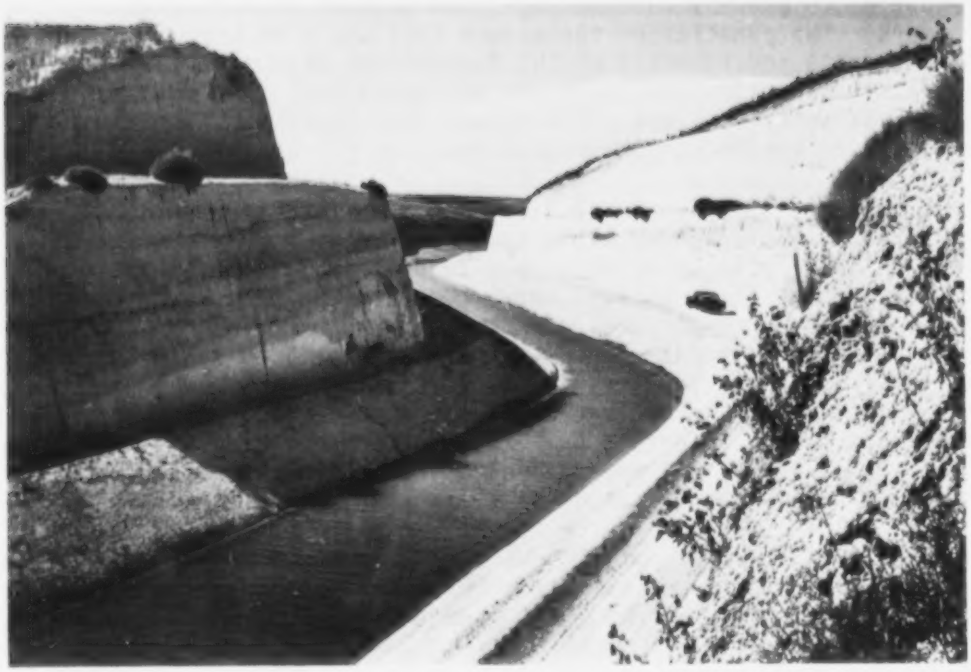


Figure 6. - Steep slopes constructed in naturally dry loess showing considerable stability (Nebraska).

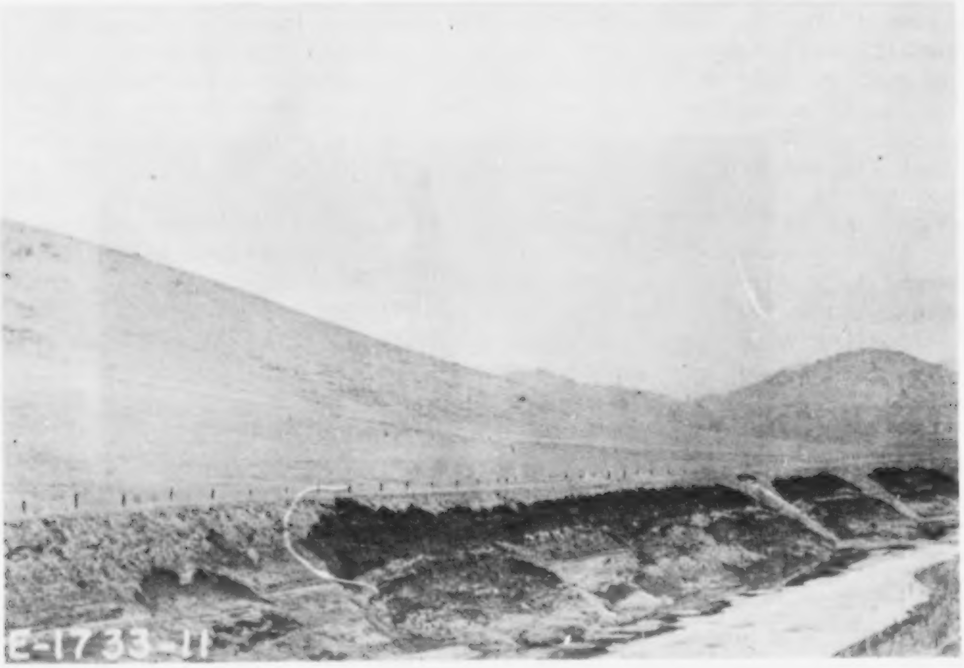


Figure 7. - Typical failure of canal slopes.

structures, it may be better to have the structure settle with the foundation to prevent underseepage than to have it supported on piles or piers. The penetration resistance test would be useful to evaluate the denseness and firmness of the foundations when more detailed laboratory consolidation tests are not warranted. The principal ways to control settlement are (1) increase footing size, (2) compact earth pads, (3) do the construction in stages, or (4) use piles.

Soil, as found in nature, has a certain amount of strength that can be destroyed by disturbance. This is very true of highly compressed clays. Figure 9 is an example of a load consolidation study in which an undisturbed sample of soil shows considerable resistance to settlement up to 20 pound-per-square-inch load, and under higher loads this natural strength breaks down. When the same soil is disturbed and recompacted, it does not have as much resistance to settlement. Loose or collapsible soils settle excessively when wetted. The best remedial measure is to collapse them by wetting before building a structure on them. The liquid limit and natural dry density criteria shown in figure 10 are useful in detecting soils which might subside.

Opposite to settlement, uplift and soil expansion may occur [2]. For example, water under head behind a foundation would have an uplifting effect. Some clays develop enormous expansion forces due to their ability to pull water into their mineral complex. For this reason, these clays are particularly objectionable in hydraulic structures. Figure 11 shows an example of soil expanding below a canal lining. Heave of the subgrade can also be caused by an excessive amount of sodium sulfate salt in the pore water that crystallizes due to temperature changes (figure 12). Heave occurred just below original ground line within 18 hours after final excavation along San Luis Drain. Bulging of the foundation cracked the concrete lining as shown in figure 12(b).

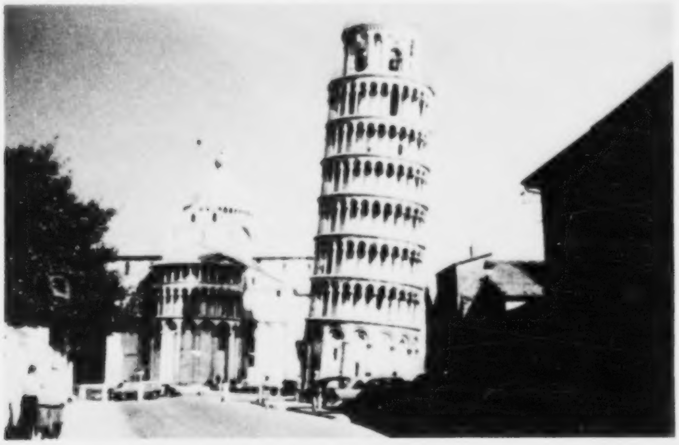


Figure 8. - Differential settlement cracked the walls and caused tilting of the Leaning Tower of Pisa.

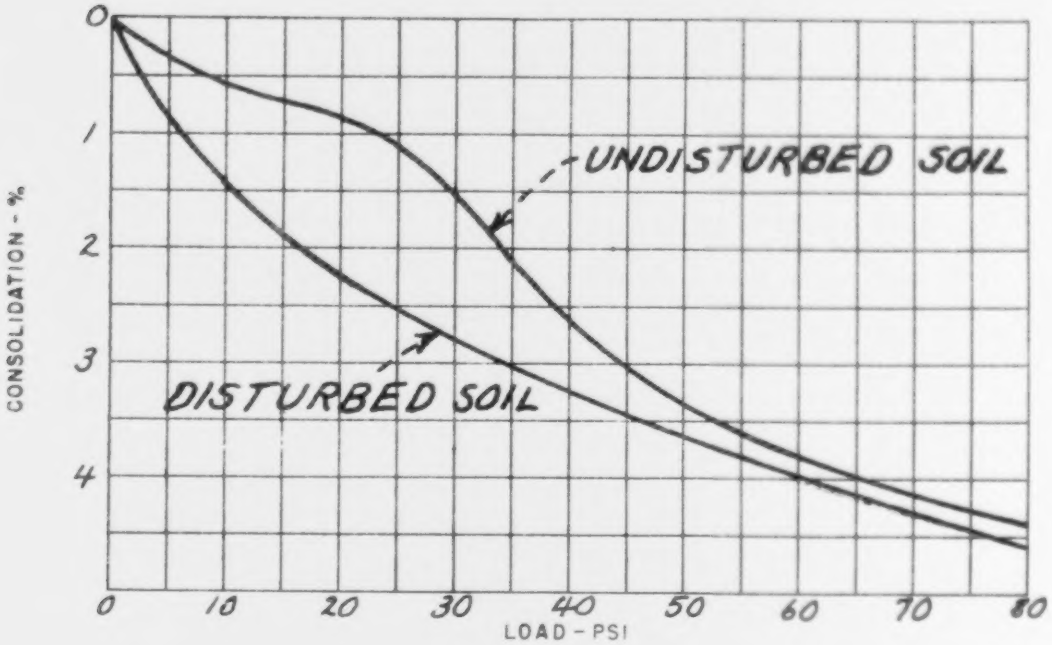


Figure 9. - Effect of disturbance on load consolidation relationship of soils.

Frost action tends to lift soils by moving water from below (by capillary action) into the freezing zone and forming ice lenses in the soil. This usually takes place in cold climates when the water table is within a few feet of the ground surface and the soils are fine-grained. Figure 13(a) shows laboratory test specimens before (No. 1) and after (No. 2) they were frozen from the top downward. The pier at the canal bottom in figure 13(b) was lifted, causing 8 inches of displacement at the pipe joint. Frost heave not only lifts the structure but permits piping after the ice lenses melt 13(c). Extensive repair work may be necessary after the spring thaw. Where necessary, frost heave can be reduced principally by (1) burying the footings below the frostline, (2) placing earth pads (sand or gravel) beneath the structure, (3) using polystyrene or some other form of insulation, or (4) draining the water to prevent buildup of ice lenses. Granular soils, rather than silts and clays, are most effective in protecting against uplift due to frost action because the larger space between particles prevents the rise of water due to capillary action and the buildup of ice lenses. However, coarse, granular soils have less value as insulation than fine-grained soils.

Freezing also affects the properties of soils when there is no source of water except that in the soil voids. When construction on an earth dam is shut down for the winter, freezing near the surface causes an increase in moisture content and a decrease in density near the surface. Snow acts as insulation and affects the depth of frost penetration. Before construction is resumed in the spring, and after frost is out of the ground, density tests need to be performed to make sure that the specification requirements for density and moisture are met.

Although frost action also affects the density and moisture of compacted earth linings after water is taken out of a canal, the overall effects which would cause serious damage and an increase in seepage do not seem to have occurred where the water table is not near the lining. Where the water table is within a few feet of the lining and especially when the canal is on a side hill where ground water can drain into the lining, then ice lenses will build up in the lining. This will reduce soil density and cause increased seepage through the lining.

This subject of frost action on linings and in the foundations of structures has been studied rather extensively in recent years. For more information on this see references /_3,4_/.

Deterioration

Some shales and dense clays lose their strength by drying and cracking and subsequent wetting. This is apparent in figure 14 showing the cracking of a laboratory sample due to drying. Therefore, it is desirable that such materials be protected from drying and deterioration before the overlying part of a structure is built. Protection can be provided by a coating (asphalt, plastic, etc.) or moist soil.

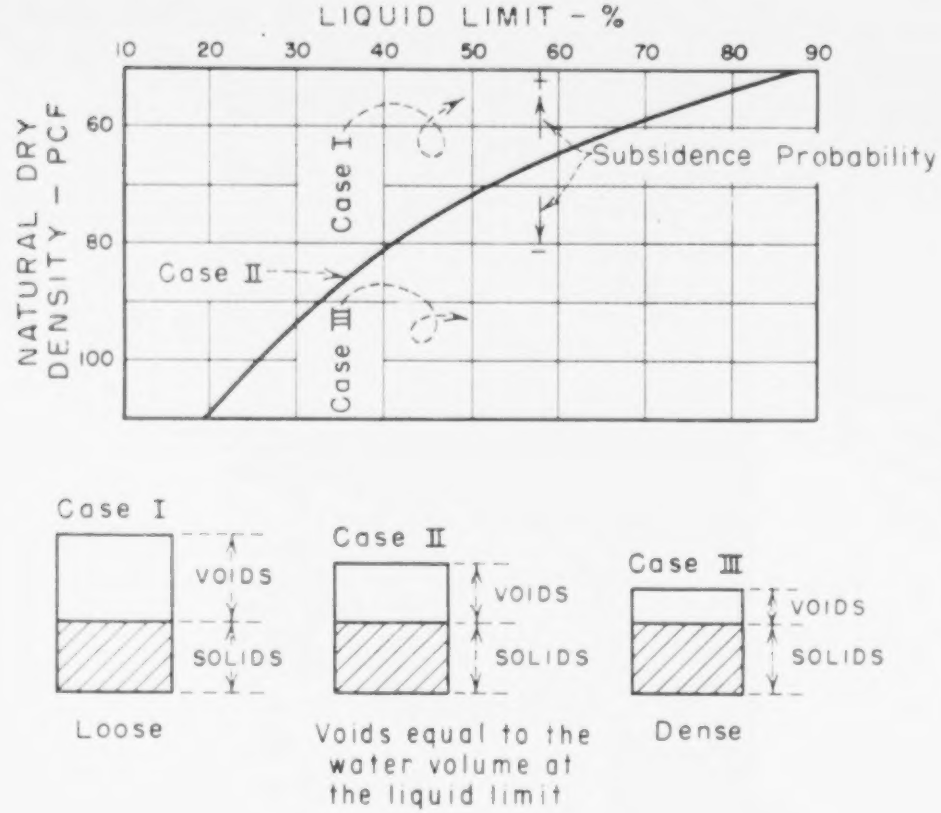


Figure 10. - Guideline for evaluating looseness and probability of subsidence.

Organic soils should not be used as a construction material because they decompose, compress, and will allow water to pass readily through them where this is not desired.

Erosion

Canal side slopes can be protected by using a gravel beach belt or a moderately plastic, compacted clay lining which falls within the limits shown in figure 15.

Permeability

Permeability is important in hydraulic structures such as those of the Bureau because much of what we build is intended to be a water barrier of some kind. In granular soils, it is surprising, however, that it takes only small amounts of silt or clay to reduce permeability. (The use chart, figure 8 of the Earth Manual, pages 20 and 21, certainly emphasizes this.) Soils such as clean gravels, sands, and some uncompacted silts have high permeability, although many, such as gravelly

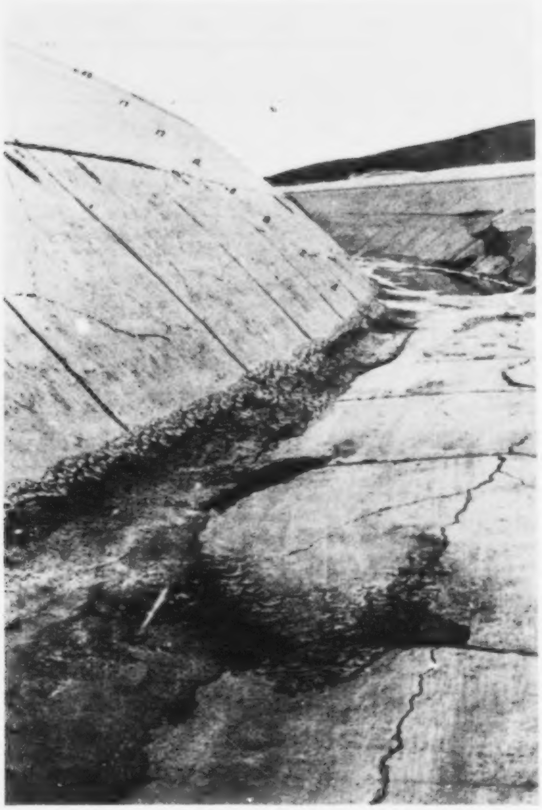
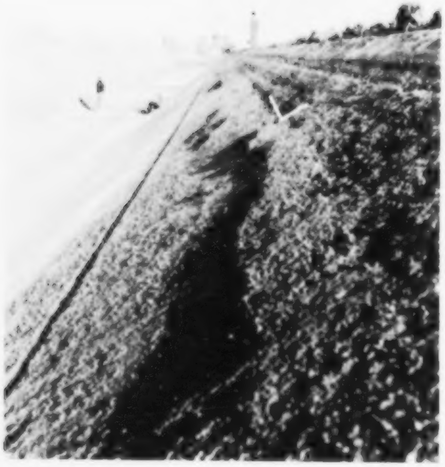


Figure 11. - Effects of swelling in expansive soil below a canal lining.

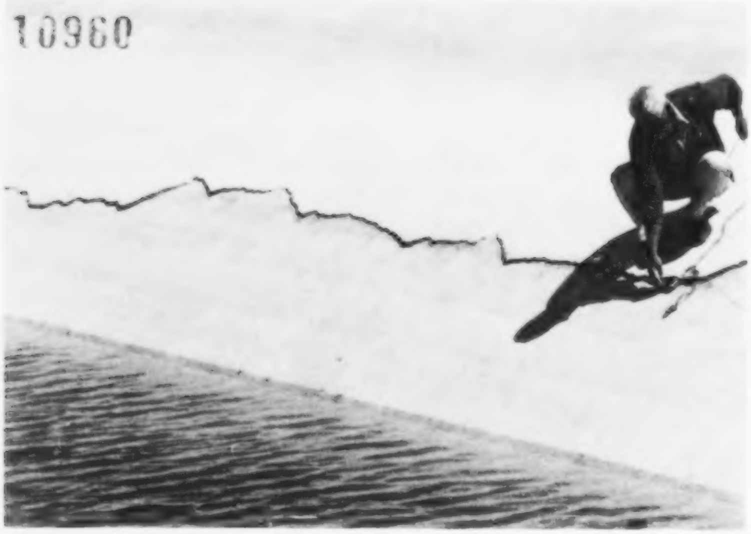
soils, may be very desirable regarding stability and low settlement. When permeability becomes excessive, water movement can remove particles and this piping action will undermine structures. In canals, piping may cause sinkholes in addition to the objectionable loss of water. There are ways of correcting the problem of high permeability and piping by blocking the movement of fine soil with layers of selected soils in a filter. However, from a soil mechanics standpoint, some clay content is the most desirable natural means of keeping permeability under control.

Considerations of Soil Types

Now that we have looked at foundations from the standpoint of general problems and requirements, let us examine various soil types, going from fine clays to coarse gravels, and discuss corrective measures that can be taken once the problem is defined. Refer to table 1 for summary. Classification of soils from laboratory tests is described in reference /̄ 5 7̄. Also soil classification is covered in the Earth Manual, test designation E-3.

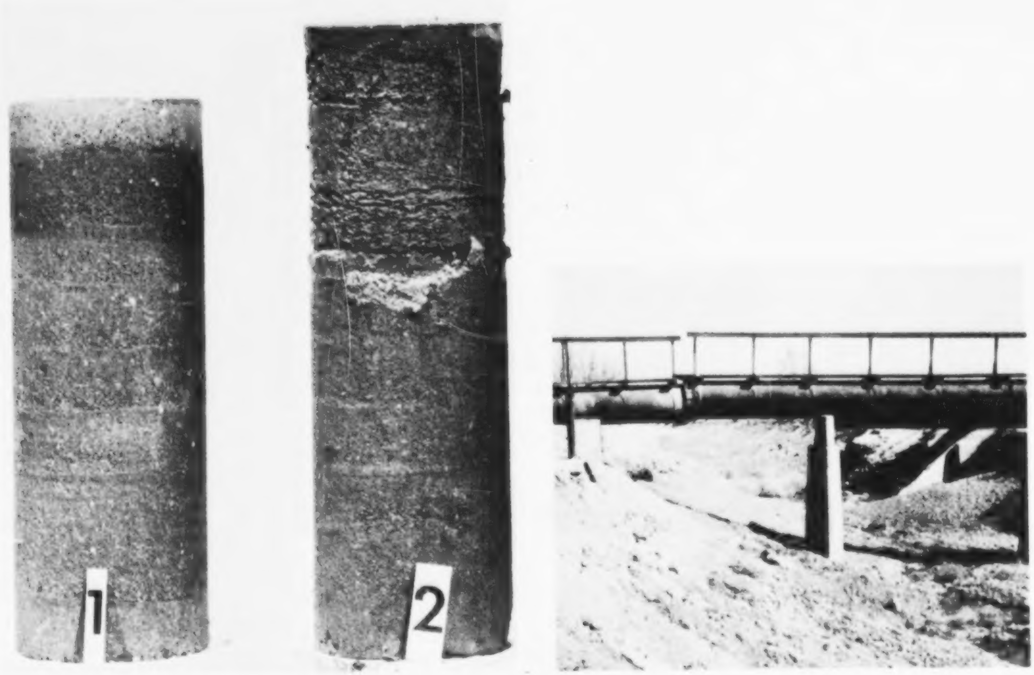


(a) Heave of 0.4 foot caused by salt crystallizing in the pore water when the temperature dropped.



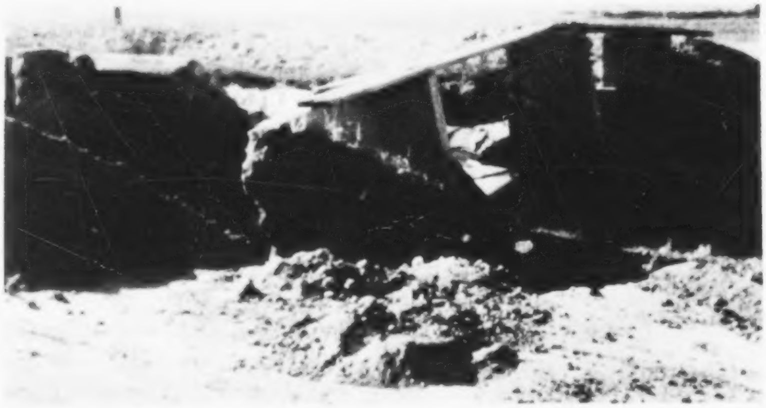
(b) Salt heave or "growing earth" phenomenon cracks concrete lining.

Figure 12. - Salt heave during construction, San Luis Drain.



(a) Frost action caused by water from below moving up into the freezing zone and forming ice lenses.

(b) Frost action raised the footing, causing 8-inch movement at pipe joint, Lower Yellowstone Project, Montana.



(c) Piping around the check structure occurred when the ice lenses melted.

Figure 13. - Frost action causes ice lenses to form which move structures and permit piping to occur.

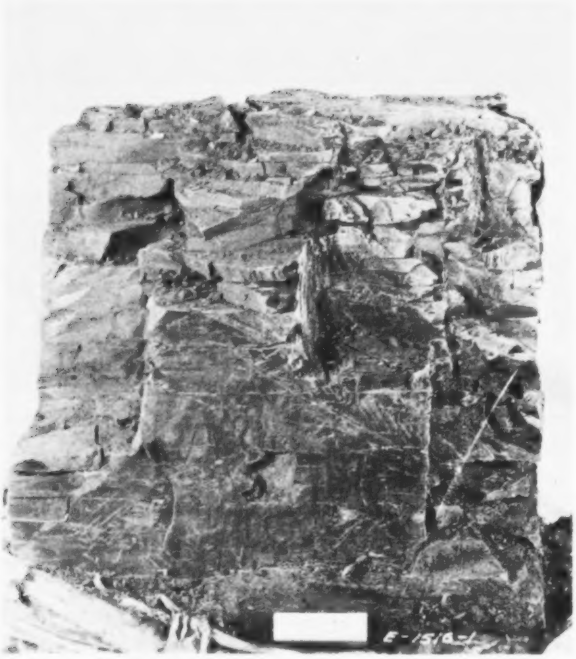


Figure 14. - Effect of air exposure and drying on shales.

Firm, Consolidated Clays

As a foundation material, firm, consolidated clays are generally desirable. They are frequently called formation material and in geologic terms may even be called bedrock. Examples of such materials are the Denver Formation in Denver and Carlyle Shale, Pierre Shale, and Colorado Shale found in the Midwest. These clays have been consolidated by large, past overlying soil or glacial ice pressures and, as a result, are preconsolidated to pressures greater than those placed on them by new structures. Also, such materials located below the ground surface are desirable for supporting piles and caissons. As mentioned previously, they may be subject to deterioration upon disturbance. However, as long as they are not disturbed, the preconsolidation pressures have given them strengths that are capable of sustaining appreciable loads. These soils may swell when the excavation process removes the overburden loads. Also, if they contain expansive clay minerals, the influx of water may make them objectionable. This will be discussed later. Nevertheless, for moderate-sized structures, the supporting capacity of such clays is usually not a problem. The location and general firmness of such clays can be determined by the penetration resistance tests.

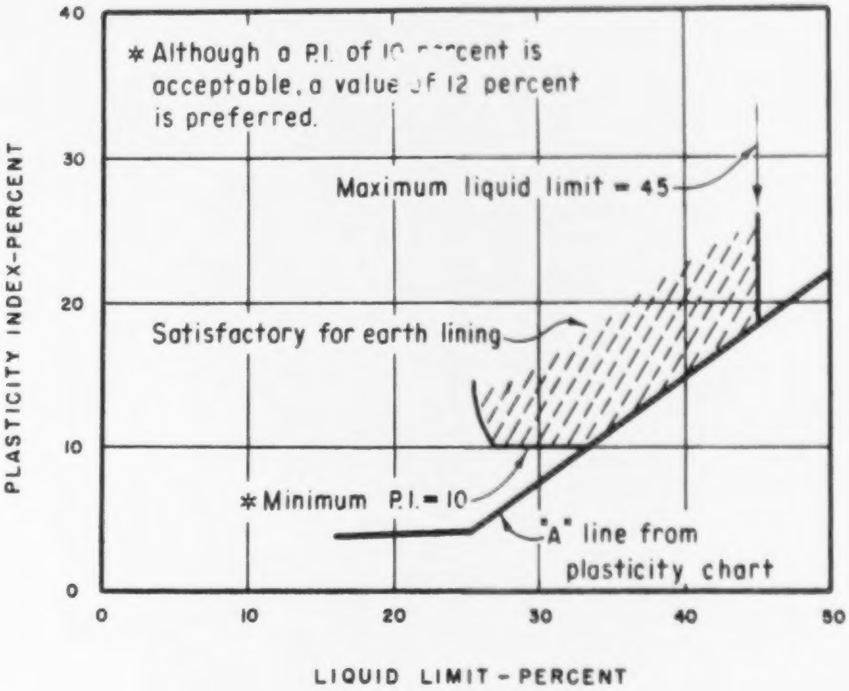


Figure 15. - Plasticity criterion for impervious, erosion-resistant, compacted earth linings.

Compressible Clays

Compressible clays are critical materials regarding foundation settlement and bearing capacity. These are the materials that require soil mechanics analyses and decisions of whether settlement can be tolerated or whether they should be improved by excavating and placing the foundation at greater depth, or by the use of piles or caissons, or use of a compacted earth pad. These clays generally have not been preconsolidated by past heavy loads. Instead, they are geologically recent deposits and, commonly, new water-deposited sediments. An extreme example is the lake sediments near Great Salt Lake, Utah. Arthur V. Watkins Dam near Ogden, Utah, was placed on this soft clay and the rather large settlements of several feet that are taking place were accounted for in the design. The dam embankment was built using the stage construction method. However, rigid structures such as the pumping plants of this project are bypassing this clay with deep piles.

Compressible clays are not necessarily as compressible as the above example but may vary up to what we call the preconsolidated or firm consolidated clays. Compressible clays are not restricted from being satisfactory foundations but are the clays which usually require tests and soil mechanics interpretation for the evaluation of their supporting capacity. They may be critical in both settlement and the problem of punching into the ground, accompanied with lateral bulging. The field penetration resistance test is a

Table 1. - Usual foundation problems and remedial measures - Summary

Foundation problem	Possible remedial measures*
CLAYS - SOFT (COMPRESSIBLE)	
Settlement	1. Reduce unit design load by increasing the footing size 2. Stage construction - gradual loading of subsoil 3. Compacted earth pad (overexcavation and refill) 4. Use piles to transfer loads to deeper soils
Slope stability	1. Flatten slope 2. Prevent saturation - install drains 3. Weight the toe of the slope 4. Unload the top of the slope 5. Strengthen soils with lime or cement additives
Erosion	1. Cover surface with erosion-resistant material 2. Densify existing soils 3. Use medium plasticity clay lining or soil-cement
CLAYS - FIRM (CONSOLIDATED)	
Expansion	1. Increase foundation loadings by using caissons embedded in nonchanging soil 2. Prevent future subsoil moisture changes - use membrane 3. Prewet foundations and retain their moisture 4. Stabilize by mixing with lime
Deterioration	1. Protect foundation surfaces from drying and cracking
SILTS AND INTERMEDIATE SOILS	
Soil collapse	1. Saturate subsoils before construction 2. Compacted earth pads
Settlement	1. Same considerations as for compressible clays
Frost action	1. Place footings below frostline 2. Restrict supply of water 3. Compact earth pad (use sand or gravel) 4. Apply thermal insulation 5. Use soil additives
SANDS AND GRAVELS	
Settlement	1. Densify subsoils using vibratory methods
Permeability	1. Mix clay with sand or gravel 2. Grout
Piping	1. Use filter zone or a clay binder
Liquefaction	1. Densify subsoils

---

* Selection of most economical remedial measure depends on the structure type and environmental considerations.

good, rapid, and preliminary method of exploring the general quality of such a foundation. Although this test shows the degree of firmness, it does not indicate settlement probability. It does, however, indicate shearing characteristics of the soil in its in-place condition, but changes in moisture may appreciably affect the strength and this effect is not indicated by an in-place test. In the case of very soft saturated clays, the vane test is a more precise method of evaluating the shearing strength of the material in place. The most acceptable evaluation of the settlement and bearing capacity characteristics is obtained from undisturbed samples and laboratory tests. When detailed tests are not practicable, in-place density and laboratory plasticity tests are very valuable to supplement the simplified field tests such as the penetration resistance.

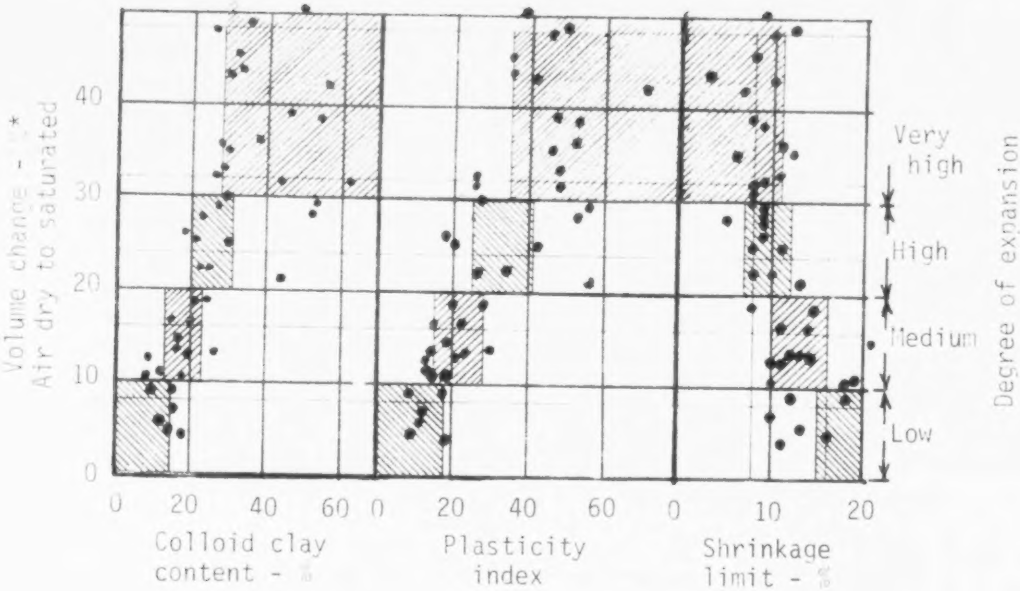
Expansive Clays [2]

The expansion of some clays when additional water is made available to them is a characteristic that is related to the type of minerals composing the clays. This can be a problem with either compressible or consolidated clays. It is, of course, far more critical when the clays are initially dense as in consolidated clays, because they then have much more possibility of expansive volume change.

These clays have mineral constituents which have a particular affinity to pull water molecules into their structure. The water effect on these clays causes them to shrink and crack when they are dried, and swell with sometimes appreciable force when they become wetted. Such clays usually fall in the CH (fat clay) classification group. Devising methods of identifying these clays has been an aid to many organizations, and we have a guideline (Earth Manual, page 212) that has received much recognition. This is shown in figure 16 and involves the use of certain standard index tests (Earth Manual test designations E-6, page 424, and E-7, page 435) on the basis of the following principles:

> A high plasticity index means that the soil can have a large change in moisture and still be in a plastic condition. Since moisture is the principal cause of volume swell, it is logical that a high plasticity index is a contributing indicator of expansion.
>
> A low shrinkage limit value means that it is possible that the soil can shrink to small volumes by drying. Therefore, this characteristic would be a contributing indicator of possible shrinking volume change.
>
> Soils with a high content of colloidal clay, which is the most active ingredient, means that there is a large amount of material which has the possibility of causing expansion.

The table shown in figure 16 is based on tests on many samples and serves as a guide to estimating the percentage and degree of expansion. In the use of this table, all three of the above-mentioned properties should be considered together to arrive at the expansion estimate.



DATA FOR MAKING ESTIMATES OF PROBABLE VOLUME CHANGES FOR EXPANSIVE MATERIALS

Data from index tests**			Probable expansion* % total volume change (dry to saturated condition)	Degree of expansion
Colloid content (% minus 0.001 mm)	Plasticity index	Shrinkage limit (%)		
>28	>35	<11	>30	Very high
20-31	25-41	7-12	20-30	High
13-23	15-28	10-16	10-20	Medium
<15	<18	>15	<10	Low

*Based on a vertical loadings of 1.0 lb/in^2.
**All three index tests should be considered in estimating expansive properties.

Figure 16. - Guidelines for predicting expansive clays from laboratory index tests.

Data in figure 16 will identify high-volume-change clays during investigations. However, during construction the in-place moisture density condition is most important. On San Luis Drain, we found that if the natural moisture of a soil was sufficiently high, it would not expand thereafter if the moisture were retained above the line shown in figure 17 /6/. If the natural clays dried during construction, they were moistened by sprinkling for 30 days before embankments were built on them. Expansive clays can be controlled from a structure foundation standpoint by (1) placing the structure on caissons to increase foundation loadings and to anchor it in nonchanging material, (2) controlling subsoil moisture changes, and (3) mixing lime with the clay to change the chemical structure and form a stronger product. Stabilization of roadways and embankments with lime on Friant-Kern Canal is shown in figure 18.

Silts and Intermediate Soils

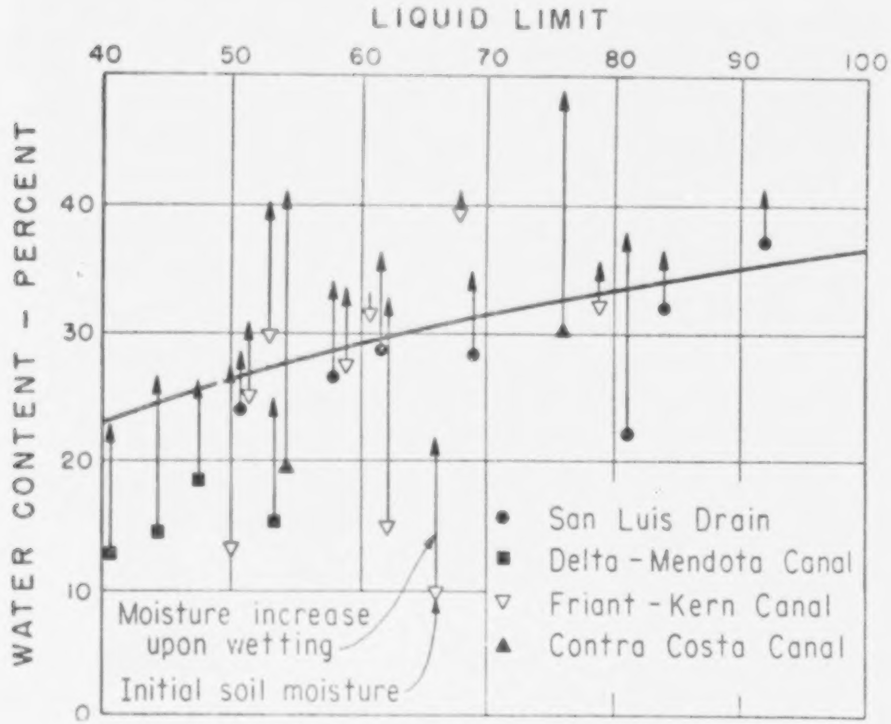
Silts and low plasticity clays are soils with moderate characteristics. They generally are not considered a problem in a recompacted condition, except they may need to be checked for settlement and shear resistance by laboratory tests. Natural deposits, however, are frequently a problem because they are sometimes very loose or of low density, such as windblown (loess) deposits of the Midwest and mudflow deposits of the San Joaquin Valley.

Silt and low plasticity clays can be very stable under dry conditions where small amounts of binder hold them together. As these soils become saturated, however, the binding effect deteriorates and the soils become unstable and frequently subside appreciably, as shown in figure 19(a). It is important that the looseness of these soils be recognized, and we feel that we have a simple procedure for doing this. Approximately 16 miles of canal subsoils were collapsed by wetting them, as shown in figure 19(b).

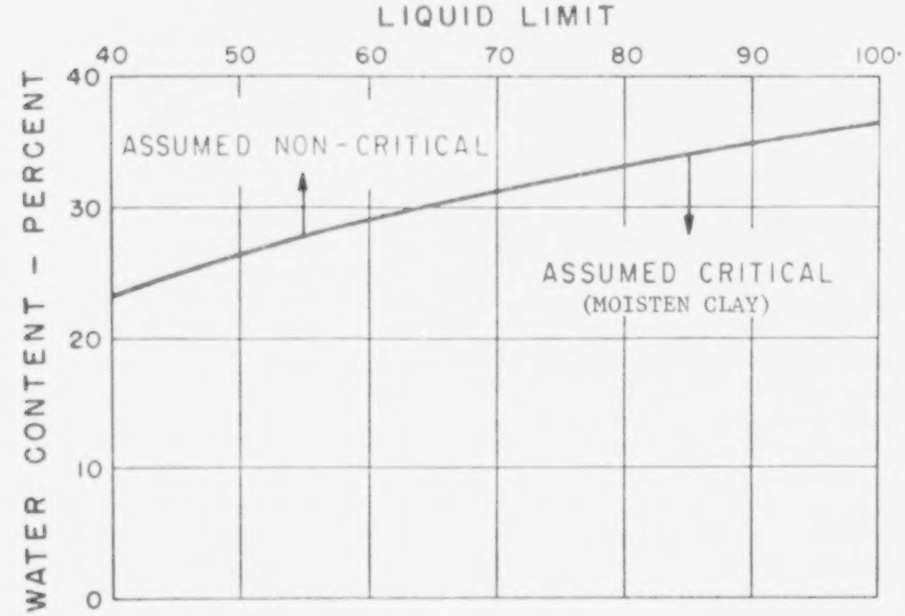
The guideline / 7 / is illustrated in figure 10. Soils of different plasticity or water-holding capacity will collapse at different densities. The liquid limit (test designations E-6 and E-7) is a moisture content, determined by standard laboratory tests, which represents the weakest plastic condition of the soil or the "approaching a liquid" condition. When the soil has a low density, such that its void space is sufficiently large to hold the liquid-limit moisture content, saturation can easily cause a liquid-limit consistency and the soil is able to collapse. When a soil has a void space too small to hold the liquid-limit moisture, it cannot reach this consistency even when it becomes saturated. It will not collapse but will retain a plastic condition and only settle as a normal result of loading.

When the natural density is evaluated at frequent locations and depths and compared to the limiting density based on the liquid-limit moisture content, the criterion discussed above becomes a useful aid in showing when in-place densities are either adequate or lower than limiting density and trend toward probable near-surface subsidence.

It is also of value to correlate the density relations to such other characteristics as degree of saturation, depth to ground water, ponding tests at representative locations, and laboratory tests on representative



(a) Increase in water content when clays expanded during laboratory tests.



(b) Minimum water content required for soil liquid limit to stabilize clays for hydraulic structure foundations.

Figure 17. - Development of moisture control criterion for reducing expansive potential of foundation clays during construction.

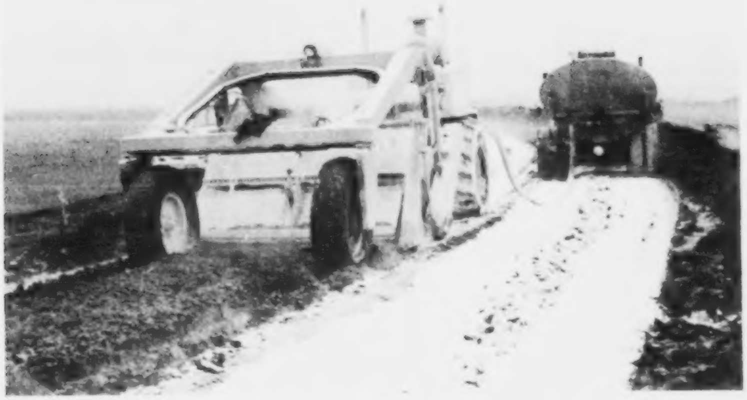


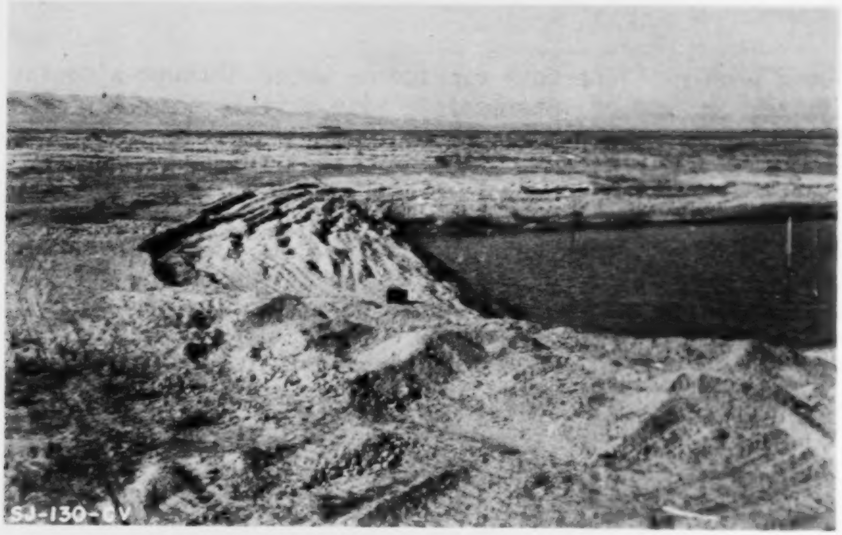
Figure 18. - Lime stabilization of roadways and clay embankments. Four percent hydrated lime was mixed with wet unstable clays (Friant-Kern Canal).

samples to evaluate loading effects. These methods were part of the San Luis Canal investigations, California, where subsidence is a serious problem.

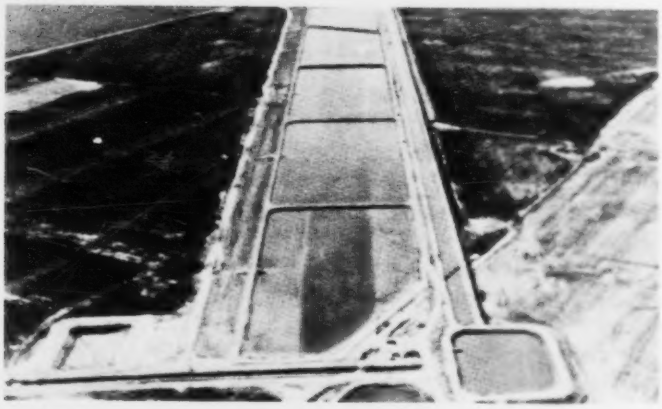
Sands and Coarse-grained Soils

Granular and gravelly soils are generally more desirable for foundations. About the only major bad feature of these soils is their occasional occurrence at low density. We use a test called relative density (Earth Manual test designation E-12, page 479) to find out how dense these soils are. This test shows the natural density in terms of the minimum and maximum density that can be obtained by a standard method in the laboratory. Normally the soil density is acceptable if it is above 70 percent relative density. If granular soils are very loose and saturated, they may liquefy under earthquake shocks.

Gravelly soils are considered particularly desirable as a construction material; they are the select materials used for roadways. The gravel has advantages in having low settlement and high shear resistance. About the only objectionable feature of gravelly soils is that of possibly high permeability. The all-around properties of gravelly soils become improved with small amounts of clay binder. Reduction of seepage in gravels is best accomplished by adding clay as a binder. However, excess clay makes the soil revert to a less stable condition. If piping is



(a) Ponding test in progress showing settling of banks.



(b) Ponding operations to consolidate the subsoils prior to the construction of canal embankments.

Figure 19. - Subsidence due to wetting of low density soils, San Luis Canal.

found to be a problem (fine soil carried by water through a coarse soil), a filter or zone of intermediate size soil is necessary to prevent the movement of soil particles.

Solid Rock Foundation

Solid rock is only mentioned here to complete the circle of earth types. It is usually considered to be the best type of foundation and has little reason to be questioned. However, rock varies in strength and sometimes is reported as bedrock when it is only a formation of consolidated clay. Also, there can be problems with some solid rocks, such as limestone which, under certain ground-water conditions, may dissolve and form potholes.

Soil Selection and Field Procedures

Personnel engaged in the operation of a water system are required to maintain, repair, and occasionally engage in relocation of existing structures or construct new structures. The majority of this work involves the use of soil as a construction material. Therefore, they should have a working knowledge of the properties of soils which will aid in the proper selection for a specific purpose. Of equal importance to proper selection is to realize and understand that the successful use of a soil depends upon proper processing, that is, to increase or decrease the moisture content as required and thoroughly mix to form a uniform homogeneous mixture, and finally, proper placement and compaction.

Soil Classification

To select soils, it is first necessary to identify and classify them according to a system which is related to their physical or engineering properties. The Bureau has adopted the Unified Soil Classification System, see designation E-3, Earth Manual. The system provides for both a visual method and a method based on laboratory tests. The proportions of the soil components (gravel, sand, silt, or clay) and the plasticity (the stickiness or cohesiveness) of the silt and clay fraction are defined. The visual method uses hand tests and visual observations and is not expected to be as precise as the laboratory method. With a limited amount of training and an interest in soil classification, one can identify soils with sufficient accuracy to classify them according to the 15 basic soil groups, which is considered to be adequate for most O&M work (see figure 20).

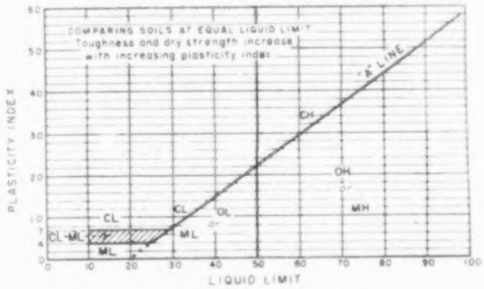
The next step in the selection of soils for the use in construction is to relate the group or type of soil and its engineering properties. This has been done on a broad basis by the "Engineering use chart" taken from the Earth Manual (see figure 21). This chart may be used as a guide for evaluating the relative desirability of the soil types for various uses and estimating their important properties. A chart listing average numerical values based on laboratory tests is shown in figure 22, which was taken from the Design of Small Dams, Second Edition, 1973.

O&M work in water systems is concerned with the repair or replacement of completed structures. Failures are usually due to the action of water.

UNIFIED SOIL CLASSIFICATION
INCLUDING IDENTIFICATION AND DESCRIPTION

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 3 inches and basing fractions on estimated weights)				GROUP SYMBOLS ℄	TYPICAL NAMES	INFORMATION REQUIRED FOR DESCRIBING SOILS	LABORATORY CLASSIFICATION CRITERIA
COARSE GRAINED SOILS More than half of material is larger than No 200 sieve size ⊠ (The No. 200 sieve size is about the smallest particle visible to the naked eye)	**GRAVELS** More than half of coarse fraction is larger than No 4 sieve size (For visual classifications, the ¼" size may be used as equivalent to the No 4 sieve size)	**CLEAN GRAVELS** (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel, max size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses. For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics. EXAMPLE: Silty sand, gravelly; about 20% hard, angular gravel particles ½ in maximum size; rounded and subangular sand grains coarse to fine; about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)	Use grain size curve in identifying the fractions as given under field identification Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No 200 sieve size) coarse grained soils are classified as follows: Less than 5% GW, GP, SW, SP. More than 12% GM, GC, SM, SC. 5% to 12% Borderline cases requiring use of dual symbols $C_u = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW
		GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures		Atterberg limits below "A" line, or PI less than 4 Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
			Plastic fines (for identification procedures see CL below)	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line with PI greater than 7
	SANDS More than half of coarse fraction is smaller than No 4 sieve size	**CLEAN SANDS** (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	SW	Well graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	SP	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW
		SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	SM	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or PI less than 4 Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
			Plastic fines (for identification procedures see CL below)	SC	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits above "A" line with PI greater than 7

	IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No. 40 SIEVE SIZE					
	DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)	GROUP SYMBOLS	TYPICAL NAMES	INFORMATION REQUIRED FOR DESCRIBING SOILS
FINE GRAINED SOILS More than half of material is smaller than No 200 sieve size **SILTS AND CLAYS** Liquid limit less than 50	None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; color in wet condition, odor if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses. For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions. EXAMPLE: Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML)
	Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
	Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity	
SILTS AND CLAYS Liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
	High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays	
	Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity	
HIGHLY ORGANIC SOILS	Readily identified by color, odor, spongy feel and frequently by fibrous texture			Pt	Peat and other highly organic soils	



PLASTICITY CHART

FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

℄ Boundary classifications - Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel-sand mixture with clay binder.

⊠ All sieve sizes on this chart are U.S. standard.

FIELD IDENTIFICATION PROCEDURES FOR FINE GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No 40 sieve size particles, approximately $\frac{1}{64}$ in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

DILATANCY (Reaction to shaking)

After removing particles larger than No 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

DRY STRENGTH (Crushing characteristics)

After removing particles larger than No 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

TOUGHNESS (Consistency near plastic limit)

After removing particles larger than the No 40 sieve size, a specimen of soil about one-half inch cube in size, is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

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Figure 20. - Unified soil classification.

103-D-347

Insofar as earthwork is concerned, an entire structure such as a canal bank or highway fill may wash out or slide, or the soil backfill around a buried structure may erode or pipe, causing partial failure. A knowledge of soils and their physical properties, particularly as influenced by water, will aid in analyzing the cause of failure and selecting the most suitable soils for repair or the proper method of processing and replacement.

Planning Earthwork Repairs or Construction

The first step in planning the repair of an earth structure or the earthwork portion of a structure is to determine or decide the basic cause of failure. The second step is to decide the best or most practical way to make the repair.

Sources of information which will aid in determining the cause of failure include: geology reports prepared during the reconnaissance, feasibility, and construction stages of the project; specifications and construction control reports: laboratory reports: prefailure conditions, such as operating conditions, weather, seismic disturbances: and finally, past observations and experience.

The type of soil, its properties, and if it has been appropriately used, should be considered when analyzing a failure. If the type and use of the soil are satisfactory, the placement conditions such as density, moisture, uniformity of material, and bond to the structure should be considered.

After inspecting the site, a decision must be made as to the best or most practical method of repair. Changes in the structure, alinement, installation of drains or drainage ditches, increase in the size or change in the type of structure, and the natural foundation or topographic features should be considered. The reuse or selection of other soil depends upon the type of soil, its properties, its present condition, intended use, and precautions required in replacement.

Basically, three classes of embankments are used in construction. Dumped fill, as the name implies, is a soil fill placed by dumping. There are no compaction or moisture requirements; hence, there are no structural requirements other than to complete the shape of an embankment or fill a hole or depression. Equipment-compacted embankments use selected material, and routing of hauling equipment provides some compaction. Moisture control may or may not be required. This type of construction is used for some minor canal embankments and road construction. The best class of embankment is rolled earthfill which requires selection of material and specific requirements as to placement density and moisture. Continuous inspection of materials and placement methods and procedures, as well as field and laboratory tests, are required to insure that specifications requirements are met. Tractor compaction is used for granular, free-draining soils in which crawler-type tractors consolidate the soil to a specified density. Rolled earthfill or tractor-compacted embankments are required when specific engineering properties are needed to insure that stability, permeability or impermeability, and minimum settlement requirements are obtained.

TYPICAL NAMES OF SOIL GROUPS	GROUP SYMBOLS	IMPORTANT PROPERTIES				RELATIVE DESIRABILITY FOR VARIOUS USES *									
		PERMEABILITY WHEN COMPACTED	SHEARING STRENGTH WHEN COMPACTED AND SATURATED	COMPRESSIBILITY WHEN COMPACTED AND SATURATED	WORKABILITY AS A CONSTRUCTION MATERIAL	ROLLED EARTH DAMS			CANAL SECTIONS		FOUNDATIONS		ROADWAYS		
						HOMOGENEOUS EMBANKMENT	CORE	SHELL	EROSION RESISTANCE	COMPACTED EARTH LINING	SEEPAGE IMPORTANT	SEEPAGE NOT IMPORTANT	FILLS		SURFACING
													FROST HEAVE NOT POSSIBLE	FROST HEAVE POSSIBLE	
WELL GRADED GRAVELS, GRAVEL SAND MIXTURES, LITTLE OR NO FINES	GW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	—	—	1	1	—	—	1	1	1	3
POORLY GRADED GRAVELS, GRAVEL SAND MIXTURES, LITTLE OR NO FINES	GP	VERY PERVIOUS	GOOD	NEGLIGIBLE	GOOD	—	—	2	2	—	—	3	3	3	—
SILTY GRAVELS, POORLY GRADED GRAVEL SAND SILT MIXTURES	GM	SEMIPERVIOUS TO IMPERVIOUS	GOOD	NEGLIGIBLE	GOOD	2	4	—	4	4	1	4	4	9	5
CLAYEY GRAVELS, POORLY GRADED GRAVEL SAND CLAY MIXTURES	GC	IMPERVIOUS	GOOD TO FAIR	VERY LOW	GOOD	1	1	—	3	1	2	6	5	5	1
WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	—	—	3 IF GRAVELLY	6	—	—	2	2	2	4
POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SP	PERVIOUS	GOOD	VERY LOW	FAIR	—	—	4 IF GRAVELLY	7 IF GRAVELLY	—	—	5	6	4	—
SILTY SANDS, POORLY GRADED SAND SILT MIXTURES	SM	SEMIPERVIOUS TO IMPERVIOUS	GOOD	LOW	FAIR	4	5	—	8 IF GRAVELLY	5 EROSION CRITICAL	3	7	8	10	6
CLAYEY SANDS, POORLY GRADED SAND CLAY MIXTURES	SC	IMPERVIOUS	GOOD TO FAIR	LOW	GOOD	3	2	—	5	2	4	8	7	6	2
INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS WITH SLIGHT PLASTICITY	ML	SEMIPERVIOUS TO IMPERVIOUS	FAIR	MEDIUM	FAIR	6	6	—	—	6 EROSION CRITICAL	6	9	10	11	—
INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	CL	IMPERVIOUS	FAIR	MEDIUM	GOOD TO FAIR	5	3	—	9	3	5	10	9	7	7
ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY	OL	SEMIPERVIOUS TO IMPERVIOUS	POOR	MEDIUM	FAIR	8	8	—	—	7 EROSION CRITICAL	7	11	11	12	—
INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	MH	SEMIPERVIOUS TO IMPERVIOUS	FAIR TO POOR	HIGH	POOR	9	9	—	—	—	8	12	12	13	—
INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	CH	IMPERVIOUS	POOR	HIGH	POOR	7	7	—	10	8 VOLUME CHANGE CRITICAL	9	13	13	8	—
ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY	OH	IMPERVIOUS	POOR	HIGH	POOR	10	10	—	—	—	10	14	14	14	—
PEAT AND OTHER HIGHLY ORGANIC SOILS	PT	—	—	—	—	—	—	—	—	—	—	—	—	—	—

* NOTE NO. 1 IS THE BEST

ENGINEERING USE CHART

Figure 21. - Engineering use chart.

Embankment Repairs or Construction

The first step in soil embankment repairs or new construction is to locate the material to be used and initiate moisture conditioning. Dry soils, particularly fine-grained soils, require time to absorb and attain a uniform moisture content. Wet, clayey soils should be worked, that is, scarified and mixed to prevent formation of hard-surfaced, dried chunks. Occasionally, blending of plastic and nonplastic soils may be required. It is preferable to mix the soils and then add moisture to insure that the final mixture is homogeneous and the moisture uniform. Proper preconditioning will overcome most of the deficiencies noted in earthwork construction.

Soil classification group	Proctor Max. dry density in lbs per cu ft	Compaction Optimum water content, percent	Void ratio, e_o	Permeability, K, feet per year	Compressibility @20 psi, percent	Compressibility @50 psi, percent	Shearing strength C_o, psi	Shearing strength C_{sat}, psi	Shearing strength tan φ
GW	>119	<13.3		27000 ± 13000	<1.4	-	-	-	>0.79
GP	>110	>12.4		64000 ± 34000	<0.8	-	-	-	>0.74
GM	>114	<14.5	-	>0.3	<1.2	<3.0	-	-	>0.67
GC	>115	<14.7	-	>0.3	<1.2	<2.4		-	>0.60
SW	119 ± 5	13.3 ± 2.5	0.37 ± -	-	1.4 ± -	-	5.7 ± 0.6	-	0.79 ± 0.02
SP	110 ± 2	12.4 ± 1.0	0.50 ± 0.03	>15.0	0.8 ± 0.3	-	3.3 ± 0.9	-	0.74 ± 0.02
SM	114 ± 1	14.5 ± 0.4	0.48 ± 0.02	7.5 ± 4.8	1.2 ± 0.1	3.0 ± 0.4	7.4 ± 0.9	2.9 ± 1.0	0.67 ± 0.02
SM-SC	119 ± 1	12.8 ± 0.5	0.41 ± 0.02	0.8 ± 0.6	1.4 ± 0.3	2.9 ± 1.0	7.3 ± 3.1	2.1 ± 0.8	0.66 ± 0.07
SC	115 ± 1	14.7 ± 0.4	0.48 ± 0.01	0.3 ± 0.2	1.2 ± 0.2	2.4 ± 0.5	10.9 ± 2.2	1.6 ± 0.9	0.60 ± 0.07
ML	103 ± 1	19.2 ± 0.7	0.63 ± 0.02	0.59 ± 0.23	1.5 ± 0.2	2.6 ± 0.3	9.7 ± 1.5	1.3 ± -	0.62 ± 0.04
ML-CL	109 ± 2	16.8 ± 0.7	0.54 ± 0.03	0.13 ± 0.07	1.0 ± 0.2	2.2 ± 0.0	9.2 ± 2.4	3.2 ± -	0.62 ± 0.06
CL	108 ± 1	17.3 ± 0.3	0.56 ± 0.01	0.08 ± 0.03	1.4 ± 0.2	2.6 ± 0.4	12.6 ± 1.5	1.9 ± 0.3	0.54 ± 0.04
OL	-	-	-	-	-	-	-	-	-
MH	82 ± 4	36.3 ± 3.2	1.15 ± 0.12	0.16 ± 0.10	2.0 ± 1.2	3.8 ± 0.8	10.5 ± 4.3	2.9 ± 1.3	0.47 ± 0.05
CH	94 ± 2	25.5 ± 1.2	0.80 ± 0.04	0.05 ± 0.05	2.6 ± 1.3	3.9 ± 1.5	14.9 ± 4.9	1.6 ± 0.86	0.35 ± 0.09
OH	-	-	-	-	-	-	-	-	-

The ± entry indicates 90 percent confidence limits of the average value
- denotes insufficient data, > is greater than, < is less than

Figure 22. - Average properties of soils.

The question is asked, how can the desirable moisture content be estimated without the use of field and laboratory tests? This is not particularly easy and requires a knowledge of the compaction characteristics of soils.

One way to judge if the moisture content of a clayey soil is near that desired for maximum compaction (optimum moisture) is to take a handful of the minus 3/16-inch fraction of the soil and squeeze it into a ball. If a firm ball with a relatively smooth surface can be formed, the soil will be near, probably slightly more than, the optimum required for standard compaction. The relatively smooth surface indicates that sufficient moisture has penetrated the clay chunks to permit them to deform.

Another method of judging the moisture content of a fine-grained silty or clayey soil or the fine-grained fraction of a coarse soil is to perform the "toughness test," one of the manual tests used in the visual method for classifying soils, see figure 20.

For silty soils, the optimum moisture for compaction is somewhat less (from 2 to about 5 percent less) than the moisture content needed for the toughness test. For lean clays, the optimum moisture and moisture for the toughness test are about the same (plus or minus about 2 percent). For fat clays, the optimum moisture is more than the toughness test, an amount ranging from 2 to 10 percent depending upon the plasticity of the clay.

It is difficult to overcome insufficient or nonuniform moisture conditions by any means other than adding moisture and thoroughly mixing prior to compaction. Occasionally, additional compaction is suggested as a substitute for moisture, that is, by increasing the number of roller passes or decreasing the thickness of lifts and obtaining the maximum or specified density at a moisture content less than the optimum desired for compaction. Insofar as permeability is concerned, laboratory tests have shown that to attain the same low permeability, the density of the soil must be greater than the maximum, when placed dry of the optimum moisture. A rule of thumb would be an increase of 1 pound in dry density for each percentage point the moisture is dry of optimum.

For coarse-grained, gravelly, sandy, or silty soils, the time element for moisture conditioning or moisture control is not critical for compaction. The moisture content should be more than the bulking moisture, which is about 5 percent, and sufficient to lubricate the particles. The adequacy of moisture can be judged by observing the material during compaction. If the fill is soft, the moisture is too great; if it is fluffy, there is insufficient moisture. If free moisture appears in the tracks immediately following passage of compaction equipment and then disappears, the moisture is adequate. Another suggestion is to observe if the soil is picked up by the tracks of the tractor and forms "cakes." If the soil is too wet or too dry, "cakes" will not form.

The second step is to prepare the foundation, abutments, and surfaces of structures. All objectionable materials such as trash, organic material, and oversized particles should be removed. Low-density, sloughed, or washed soils should be removed, or stockpiled if the moisture is to be reused. During the latter operation, moisture conditioning should be initiated, adding moisture or drying as required. The purpose of the cleanup is to remove undesirable natural foundation or uncompacted materials and expose competent surfaces on and against which to place compacted soil.

The lowest part of the foundation should be leveled. If the foundation is rock or surfaces of concrete structures are exposed, they should be clean (free of loose or chunks of natural material) and moistened by spraying before placing soil on or against them. If the exposed foundation is clayey soil, the natural moisture should be maintained by spraying. If this is not practicable, the dried surface material should be removed or scarified, moistened, thoroughly mixed, and compacted. If the foundation is sandy or granular silty soils, the foundation should be thoroughly moistened before placing the first layer of soil. Soil should not be placed on any foundation having standing or flowing water on the surface. Soils should not be placed when either the soil or the surface on which the soil is to be placed is frozen.

As construction proceeds, weathered or dried material on the exposed abutments or side slopes should be removed and the surfaces moistened. Structure or rock surfaces should be moistened.

The third step is to construct the embankment /̲ 8, 9, 10 /̲. The preconditioned soil should be placed, thoroughly mixed, and spread to the desired thickness to form a homogeneous mass free from lenses, pockets, streaks, or layers of material differing substantially in texture or gradation or moisture from the surrounding material and, when compacted, free of potential paths of percolation.

If neccessary, moisture should be added to the material dried during the final mixing. In embankment construction, the coarser particles should be placed toward the outer slopes. An effort should be made to maintain the surface of each compacted layer level and horizontal. The surface of the preceding layer must be scarified and correctly moistened or dried before placing the next layer.

The next step in construction is to compact or densify the loose layer of soil as required to obtain stability against sliding, to reduce the permeability through the soil mass, to obtain and maintain good bond between the compacted soil and surfaces of structures, and to minimize settlement.

The sheepsfoot tamping roller is the most desirable method for compacting clayey or silty soils. Bureau experience has shown that about 12 passes of the standard Bureau roller, which weighs 4,000 pounds per linear foot, will produce a density equivalent to the maximum obtained by the standard Bureau laboratory compaction test, designation E-11, Earth Manual, providing the soil is near (plus or minus 2 percentage points) the optimum moisture, and the thickness of the layer after compaction is 6 inches (8 to 9 inches thick before compaction). The maximum size of particles in the soil to be compacted is limited to about 5 inches.

Therefore, if the standard roller is available and the other items of moisture and thickness of lift are controlled, 12 passes will produce within tolerable limits the desired density.

It should be noted that for equivalent compaction the weight of the roller, number of passes, and thickness of layer are interrelated. However, 24 passes of a roller weighing 2,000 pounds per linear foot will not produce the same degree of compaction as 12 passes of a 4,000-pound-per-linear-foot roller on the same thickness of lift. Rollers usually "walk out" after the density they are capable of producing is attained and further rolling produces little or no additional increase in density. (Counting the number of blows to drive an eightpenny nail using a regular or lightweight tack hammer could be considered a comparable illustration. Depending upon the weight of the hammer, it may be impossible to drive the nail regardless of the number of blows.)

If a tamping roller weighing less than 4,000 pounds per linear foot is the only roller available, the thickness of lift should be reduced proportionally; 3-inch compacted lift or 4-inch loose lift would be required for 12 passes of a roller weighing 2,000 pounds per linear foot to maintain equivalent compactive effort and, hence, attain approximately equivalent density. The number of passes may be reduced if the roller has definitely "walked out." Reducing the thickness of lift also reduces the maximum size of particle permitted in the soil to be compacted. In case of a 3-inch compacted lift, the maximum size of particle should be about 2-1/2 inches.

Sometimes "hardness" and "density" are confused when evaluating the results of compaction. The expression that the soil is "packed hard" does not always indicate that it is dense. If the moisture of a clayey or silty soil is less than the optimum required for compaction, the surface may become very hard during compaction and yet the density could be low. Clayey soils are usually plastic and give the impression of being soft, at least not hard, when compacted at the optimum moisture content; however, depending upon the amount of coarse grains in the soil, they may become firm when compacted.

To consolidate most granular, free-draining soils, Bureau experience has shown that four passes of a crawler-type tractor weighing about 40,000 pounds will produce a density equivalent to about 70 percent relative density (see designation E-12, Earth Manual) providing the soil is thoroughly wetted and the thickness of layer after compaction is no more than 12 inches, the maximum size of particle in the soil to be compacted being limited to about 12 inches.

The area of the treads and weight of the tractor influence the pressure imposed on the soil. A review of manufacturers' data indicated that heavy tractors weighing from 40,000 to 58,000 pounds exerted a soil pressure of about 0.6 ton per square foot, those weighing from 10,000 to 26,000 pounds have soil pressure of about 0.5 ton per square foot, and the lightweight tractors weighing about 7,000 pounds have soil pressures of about 0.4 ton per square foot.

To obtain densities comparable to those obtained with four passes of a heavy tractor, the thickness of lift and maximum size of particle should be limited to about 8 inches for the medium tractor and 6 inches for the smaller sized tractor.

Vibrating tamping rollers and vibratory compactors are being used to compact silty and clayey soils as well as coarse- to fine-grained, free-draining soils. Manufacturers claim that these rollers are more efficient than the conventional methods for compaction or consolidating soils. The Bureau has not established guidelines based on weight, number of passes, or thickness of lift for the use of vibrating rollers. These rollers are being permitted on some Bureau construction and their performance is closely checked by field and laboratory tests. Moisture control and preconditioning of the soil are just as important when using vibrating rollers as conventional tamping rollers.

Backfill for Repairs or Construction

Backfill is a term used to define earth placement in confined spaces or specified within certain boundaries. There are two classes: backfill comparable to dumped embankment, in which there are no compaction requirements such as the soil used to fill trenches above pipes, in depressions, around tower footings, or to cover siphons; and compacted backfill and consolidated backfill comparable to the best class of construction for rolled earthfill and tractor-compacted granular embankments.

The steps discussed for planning and performing embankment repairs or construction apply equally or to a greater degree to compacted or consolidated backfill. Backfill is usually more critical than embankment construction because of the small amount of soil required and limited working space, which limits the size of compaction equipment. Backfill around structures, particularly hydraulic structures, is especially critical because of the bond required between the soil and very dissimilar materials, such as concrete, steel, rock, or asphalt, and also because of the relatively smooth and sometimes overhanging surfaces encountered. A localized failure may cause complete failure of a structure because there is limited possibility of adjustments to take up local weaknesses in bond or deficiency in density. Therefore, the requirements for preconditioning the soil, proper preparation of foundation and abutments, and attaining adequate compaction are extremely important for compacted or consolidated backfill.

Because of limited working space, densification of backfill is done by hand-operated power tampers and surface or internal vibrators. The maximum size of particles permitted is usually limited to 3 inches. Conventional methods and equipment for compacting and consolidating backfill are discussed in the following paragraphs.

Backfill compaction of silty and clayey soils is usually done by hand-operated power tampers. Tests conducted using different types of tampers showed that the total weight and area of tamping foot influenced the number of passes required to densify a 6-inch-thick compacted layer (8-inch loose thickness) to the maximum standard laboratory density. For example, a tamper weighing 220 pounds with a tamping foot of 75 square inches required about 150 percent coverage (1-1/2 minutes per square yard of surface for most commercial tampers) to produce the maximum density; whereas, a tamper weighing 31 pounds with a tamping foot of 25 square inches required about 450 percent coverage (4-1/2 minutes per square yard of surface). The time required is dependent upon the frequency of impact, which in turn depends upon the equipment being in good operating condition and the air pressure of pneumatic equipment is as specified by the manufacturer. This study led to performance specifications for backfill tampers comparable to the 12 roller passes for the standard tamping roller, providing, of course, all other requirements for preconditioning materials, thickness of lift, and operating conditions of the equipment are met. The specifications are as follows:

> "The degree of compaction shall be equivalent to that obtained in a layer of material 6 inches thick, after compaction, by using a pneumatic tamper operating on an air pressure of not less than 100 pounds per square inch; having a total weight in pounds, including the weight of the tamping foot, of not less than 2-1/2 times the area, in square inches, of the tamping foot; and operating uniformly over an area of 1 square yard for a period of not less than 1-1/2 minutes. Provided, that the area of the tamping foot shall not be less than 40 square inches. A reduction in size of the tamper can be compensated for by a reduction in thickness of the compacted layer."

It is suggested that compaction be done in terms of passes: the tamper should be moved continuously to cover the area (one pass) and then the procedure should be repeated for a second pass. This will require about 2-1/2 minutes per square yard. Note that "a reduction in size of tamper can be compensated for by a reduction in thickness of the compacted layer." This provision is comparable to that suggested for rollers with less weight than the standard.

Backfill consolidation of granular free-draining soils is usually done by hand-operated, surface and internal vibrators. The Bureau has conducted a cursory study of the performance characteristics of surface vibrators, and a guide for determining the adequacy of compaction in lieu of tests can be suggested at this time. The specifications are as follows:

> "Compacting cohesionless materials. - The distribution of the materials shall be such that the compacted material will be homogeneous and free from lenses, pockets, streaks, or other imperfections. During compaction operations, the materials shall be wetted thoroughly, as determined by the contracting officer, so as to produce the maximum density that can be obtained by the method of compaction specified.
>
> "The degree of compaction shall be equivalent to that obtained in a layer of material, 6 inches thick after compaction, by using a flat plate-type surface vibrator with a baseplate area of not less than 300 square inches, having a total weight in pounds not less than half the vibrator baseplate area in square inches and a centrifugal (vertical impact) force of at least 4 pounds per square inch delivered at a rate of 5,000 vibrations per minute (r/min) at the baseplate, and operated uniformly over an area of 1 square yard for a period of not less than 1 minute. The thickness of the compacted layer may be increased at the discretion of the contracting officer provided a larger and more efficient compactor is used."

Internal vibrators (immersion-type concrete vibrators) have proven to be particularly successful in consolidating pipe backfill. The procedure is to place the soil to be consolidated to a depth equal to the length of the operating head of the vibrator or about 1 foot.

Water is applied and the soil is kept saturated while the vibrators are immersed the full length of the operating head, so as to penetrate the preceding layer 1 or 2 inches, and slowly withdrawn.

Penetrations spaced at about 1-foot centers for a 6-inch-diameter vibrator and about 9 inches for a 3-inch vibrator, allowing about 15 seconds for penetration and withdrawal, will produce consolidation to about 70 percent relative density. For this method of consolidation, the maximum size of particle should be limited to about 1-1/2 inches.

Puddling is another method used primarily for backfilling buried pipe. Briefly, the procedure is to deposit the soil in a pond of water about 1 foot in depth, adding soil until all the free water is absorbed. The layer is then worked vigorously with shovels or other approved tools. Another pool of water is formed and the process is continued until the backfill is completed. Sandy or silty soils are preferable for this method, although sandy lean clays are sometimes used. Puddled backfill may be satisfactory when settlement or stability is not important and when an impervious backfill, including the bond to a structure surface as required, is attained without high density. The advantages of this method are that it is easy to place in restricted areas and it thoroughly wets the foundation which helps to settle low-density foundations. The disadvantage is the low density. A density of only 80 percent of Proctor standard compaction has been reported for silts. Poor stability and possibility of large settlement and shrinkage result. The latter could lead to piping along the surfaces of the structure and through the soil. Another method using granular soil is sluiced backfill, sometimes used for cut and cover sections when there are not structural requirements for the backfill.

In instances where wet, highly plastic clays are encountered in earthwork repairs, consideration may be given to the addition of 3 to 6 percent lime to assist in reducing both plasticity and moisture content. This tends to improve workability during compaction and ultimately the stability of the repaired area.

Repair or Construction of Other Earthwork

Other earthwork repair or construction involves the same basic considerations discussed for embankment and backfill. Many of these special purpose uses involve restrictions on the type of soil and are discussed in the *Earth Manual*. Some of the uses encountered in O&M work will be discussed briefly.

Compacted earth lining may be constructed of selected coarse- or fine-grained impervious soils, such as GC (clayey gravel), SC (clayey sand), CL (lean clay), CH (fat clay), or GM (silty gravel). Soils of these types often do not require erosion protection. The coarse-grained soils are erosion resistant because of the large particles, and the fine-grained clayey soils because of plasticity. Fine-grained clayey soils must have a plasticity index of at least 10 to provide adequate resistance to erosion. SM (silty sand) and ML (silt) may be sufficiently impervious for lining material but cover must be provided for erosion protection. Uniformity of material and moisture is particularly important when materials are blended to construct a lining. Maintaining the moisture near or just wet of optimum and obtaining good bond between layers are required to insure a satisfactory lining.

Filters are used to carry away seepage water without erosion or displacement of the soil. They are constructed of specially selected pervious soils, processed, if necessary, to meet rigid gradation requirements. These requirements and precautions in construction are discussed in the Earth Manual.

Protective covers are layers of selected pervious material for preventing the displacement of soil particles by water flowing through or past the soil surface. The maximum size of particle and grading depend upon the purpose. Riprap is a form of protective blanket where the particles range in size from 1 cubic yard to 1 cubic foot. The maximum size depends upon the severity and anticipated height of waves in case of dams and the expected maximum velocity of water in case of canals. Bedding for riprap is another type of protective blanket which is equally important in providing protection. For canals, the bedding generally consists of selected sand and gravel reasonably well graded, having a maximum size of about 1-1/2 inches and less than 5 percent fines. For dams, it consists of gravel and cobbles 3/16 inch to 3-1/2 inches. Failures may be the displacement of the riprap particles down the slope or along the canal and displacement of the soil beneath the blanket. The first is generally due to improper size and grading of riprap particles. The second is generally due to improper grading of the bedding, permitting the foundation to erode, and the riprap particles to settle. When a failure has occurred, the cause should be determined before repairs are initiated.

A cover layer used for erosion protection of unlined or earth-lined canals is a form of protective blanket, consisting of pit-run sand and gravel up to 3 inches in size and preferably with a maximum of 5 percent fines. The range in grading recommended for reasonably well-graded material is shown in figure 112, page 273 of the Earth Manual.

The primary consideration in placing protective blankets is to minimize segregation and loose pockets. Adding moisture to sand and gravel mixtures will minimize segregation. Although riprap is usually placed by dumping, an effort should be made to see that the individual loads contain a good representation of all particle sizes.

Filters

For drains, a granular soil is usually used in a trench or blanket and such soil is called a filter material. Perforated pipe or pipe laid with open joints is often placed in the filter material where a large amount of water is to be drained. The filter material needs to be coarse and pervious enough to carry off the water that comes from the surrounding foundation soil, which is called the "base material" but fine enough so it will not become plugged with soil fines and stop the drainage. Definite rules have been drawn up to show what the gradation of the filter material should be for a particular base material. (See Earth Manual, pages 305 to 309.) Sometimes a two-layer filter, called a zoned filter, is necessary for water to flow from a fine sandy soil to openings in pipe without fines plugging the voids. Usually a sand within the grading limits specified for a concrete sand is a good material for the first layer placed against a fine soil / 11 /.

Pipe Bedding and Backfill

The following is quoted from the preface and introduction of a recent training manual / 12 / on pipe bedding and backfill:

"This manual presents the requirements of the Bureau of Reclamation for constructing bedding and backfill for buried pipe. Many of these requirements are new. All Bureau personnel involved in installing buried pipelines should become familiar with these requirements and the reasons they are important."

The proper installation of buried pipe is being emphasized because of:

1. The present trend to use pipelines rather than canals. - Pipelines provide more farmland use, reduce evaporation and maintenance, and are safer for the public and livestock.
2. The use of larger pipe. - Each pipe unit becomes more expensive and special installation requirements are necessary to prevent failures.
3. The availability of new pipe types and materials. - Some of these may require special installation requirements.
4. Problems with installations. - Recent experience in pipe installation has shown that new requirements are necessary.

The Bureau has recently completed a comprehensive review and revision of the specifications paragraphs and drawings for pipe installation. The discussions in this manual on bedding and backfill requirements reflect these revisions and pertain only to specifications issued after the summer of 1978. However, many of the concepts presented are also applicable for prior specifications.

The material in this manual is for pipe with diameters of 108 inches or less. Pipe larger than this size usually require special designs and provisions for bedding and backfill.

Pipelines that must be located on the sides of steep slopes also present special design and construction problems that vary with local conditions. These special cases are not covered in this manual.

Proper pipeline installation involves much more than just covering up the pipe. A buried pipe is a structure that incorporates both the properties of the pipe and the properties of the soil surrounding the pipe. The structural design of a pipeline is based on certain soil conditions, and construction control is important to assure these conditions are met.

There are two basic types of pipe, rigid and flexible. Rigid pipe must be supported on the bottom portion of the pipe. Flexible pipe must be supported on both the bottom and on the sides of the pipe.

Proper soil support of the pipe is critical to its performance, and proper inspection of the pipe installation is essential in obtaining the required support.

Inspection for proper soil support involves checking the:

1. Adequacy of soil in trench walls and foundation
2. Type of soil used for bedding and backfill
3. Distribution of soil around pipe
4. Density of soil around pipe
5. Deflection of pipe (flexible)

It is important to note that the installation requirements are different for each of the following cases:

1. For rigid pipe
2. For flexible pipe
3. For 10-inch diameter pipe and smaller
4. For pipe with diameters between 12 and 54 inches
5. For pipe with diameters between 60 and 108 inches
6. For PVC pipe

Bureau Publications on Related Earthwork Subjects

	Title	Laboratory EM Report No.
1.	The Use of Laboratory Tests to Develop Design Criteria for Protective Filters	425
2.	Experiences with the Consolidation of Pipe Bedding by Vibration on the San Diego Aqueduct	461
3.	Thick Compacted Earth Linings for Canals	462
4.	Compaction Characteristics of Gravelly Soils	509
5.	Soils Engineering in Frost Areas	510
6.	Expansive Clays - Properties and Problems	568
7.	Performance of Compacted Loessial-Soil Canal Linings	575
8.	Laboratory Filter Tests on Concrete Sand and Gravel	588
9.	Earth Manual, Second Edition, 1974	
10.	Moisture-Density Relation of Soils - Geotechnical Training Manual No. 1 (1979)	
11.	Earthwork Construction Control - Earth Sciences Training Manual No. 2 (1976)	
12.	Laboratory Classification of Soils - Earth Sciences Training Manual No. 4 (1977)	

REFERENCES

/ 1 / Earth Manual, Second Edition, U.S. Bureau of Reclamation, 1974

/ 2 / Holtz, W. G., "Expansive Clays - Properties and Problems," Report EM-568, Bureau of Reclamation, Denver, Colorado, Nov. 1959

/ 3 / Jones, C. W., D. G. Miedema, and J. S. Watkins, "Frost Action in Soil Foundations and Control of Surface Structure Heaving," Report No. REC-ERC-82-17, a report of a team of frost action, June 1982

/ 4 / Jones, C. W., "Closed-System Freezing of Soils in Linings and Earth Embankment Dams" Report No. REC-ERC-81-1, March 1981

/ 5 / Howard, A. K., "Laboratory Classification of Soils" - Earth Sciences Training Manual No. 4, 1977

/ 6 / Bara, J. P., "Controlling the Expansion of Dessicated Clays During Construction," Second International Research and Engineering Conference on Expansive Soils, Texas A&M University, Aug. 1969

/ 7 / Bara, J. P., and H. J. Gibbs, "Predicting Surface Subsidence from Basic Soil Tests," American Society for Testing and Materials, Special Technical Publication No. 322, 1962

/ 8 / Howard, A. K., "Earth Work Construction Control" - Earth Sciences Training Manual No. 2, 1976

/ 9 / Howard, A. K., "Rapid Method of Earth Work Construction Control" - Geotechnical Earth Sciences Training Manual No. 3, 1976

/ 10 / Howard, A. K., "Moisture-Density Relation of Soils" - Geotechnical Training Manual No. 1, 1979

/ 11 / Jones, C. W., "Laboratory Filter Tests on Sand and Gravel," Report No. EM-588, Bureau of Reclamation, Denver, Colo. Aug. 1960

/ 12 / Howard, A. K., "Pipe Bedding and Backfill" - Geotechnical Training Manual No. 7, 1981

APPENDIX

DEFINITIONS OF SOME COMMON SOILS AND ROCK ENGINEERING AND PIPE TERMS

BEDROCK (ledge) - Rock of relatively great thickness and extent in its native location.

BENTONITIC CLAY - A clay with a high content of the mineral montmorillonite, usually characterized by high swelling on wetting.

CAPILLARY ACTION (capillarity) - The rise or movement of water in the void spaces of a soil or rock due to capillary (suction) forces.

COHESION - Shear resistance at zero normal stress (an equivalent term in rock mechanics is intrinsic shear strength).

COHESIONLESS SOIL - A soil that when unconfined has little or no strength when air-dried and that has little or no cohesion when submerged.

COHESIVE SOIL - A soil that when unconfined has considerable strength when air-dried and that has significant cohesion when submerged.

COLLOIDAL PARTICLES - Particles that are so small that the surface activity has an appreciable influence on the properties of the aggregate.

COMPACTION - The densification of a soil by means of mechanical equipment.

COMPACTION CURVE (Proctor curve) (moisture-density curve) - The curve showing the relationship between the dry unit weight (density) and the water content of a soil for a given compactive effort.

COMPACTION TEST (moisture-density test) - A laboratory compacting procedure whereby a soil at a known water content is placed in a specified manner into a mold of given dimensions, subjected to a compactive effort of controlled magnitude, and the resulting unit weight determined. The procedure is repeated for various water contents sufficient to establish a relation between water content and unit weight.

COMPRESSIBILITY - Property of a soil or rock pertaining to its susceptibility to decrease in volume when subjected to load.

CONSOLIDATION - The gradual reduction in volume of a soil mass resulting from an increase in compressive stress.

CRITICAL HEIGHT - The maximum height at which a vertical or slope bank of soil or rock will stand unsupported under a given set of conditions.

FILL - Manmade deposits of natural soils or rock products and waste materials.

FILTER (protective filter) - A layer or combination of layers of pervious materials designed and installed in such a manner as to provide drainage, yet prevent the movement of soil particles due to flowing water.

FINES - Portion of a soil finer than a No. 200 (75-micrometer) U.S. standard sieve. (A No. 200 size particle is about the smallest particle that can be distinguished by eye.)

FOOTING - Portion of the foundation of a structure that transmits loads directly to the soil.

FOUNDATION - Lower part of a structure that transmits the load to the soil or rock.

FOUNDATION SOIL - Upper part of the earth mass carrying the load of the structure.

FROST ACTION - Freezing and thawing of moisture in materials and the resultant effects on these materials and on structures of which they are a part or with which they are in contact.

FROST HEAVE - The raising of a surface due to the accumulation of ice in the underlying soil or rock.

LIQUEFACTION (spontaneous liquefaction) - The sudden large decrease of the shearing resistance of a cohesionless soil. It is caused by a collapse of the structure by shock or other type of strain and is associated with a sudden but temporary increase of the pore-fluid pressure. It involves a temporary transformation of the material into a fluid mass.

LIQUID LIMIT, LL - The water content at which a pat of soil, cut by a groove of standard dimensions, will flow together for a distance of 1/2 inch (12.7 mm) under the impact of 25 blows in a standard liquid limit apparatus.

MOISTURE CONTENT - The percentage by weight of water contained in the pore space of a rock or soil with respect to the weight of the dry solid material.

PERMEABILITY - The capacity of a soil or rock to conduct water or other fluids.

PIPE TERMINOLOGY - The crown is the inside top of the pipe. The invert is the inside bottom of the pipe. The spring line is the horizontal line at the halfway point of the pipe. The haunches of the pipe are the outside areas between the spring line and the bottom of the pipe.

PIPING - The progressive removal of soil particles from a mass by percolating water, leading to the development of channels through the soil.

PLASTICITY - The property of a soil or rock which allows it to be deformed beyond the point of recovery without cracking or appreciable volume change.

PLASTIC LIMIT, PL - Water content at which a soil will just begin to crumble when rolled into a thread approximately 1/8 inch (3.2 mm) in diameter.

ROCK - Any naturally formed aggregate of mineral matter occurring in large masses or fragments.

SEEPAGE - The infiltration or percolation of water through rock or soil to or from the surface. The term seepage is usually restricted to the very slow movement of ground water.

SHRINKAGE LIMIT, SL - The maximum water content at which a reduction in water content will not cause a decrease in volume of the soil mass.

SOIL (earth) - Sediments or other unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks, and which may or may not contain organic matter.

TRENCH - Usually a long, narrow, near vertical-sided cut in rock or soil such as is made for utility lines.

UNIT WEIGHT - Weight per unit volume.

VANE SHEAR TEST - An in-place shear test in which a rod with thin radial vanes at the end is forced into the soil and the resistance to rotation of the rod is determined.

Water Systems Operation and Maintenance Workshop - 1990

CORROSION CONTROL BY COATINGS AND CATHODIC PROTECTION - PART 1

Session 11-1

Owen Uhlmeyer

Introduction

Cathodic protection!!! Sounds rather mysterious, doesn't it? Like some sort of old-age, social security program or perhaps some private policing agency. However, such is not the case and during this period we shall attempt to take some of the mystery out of cathodic protection by providing answers to some of the questions you may have formulated regarding this subject. Basic questions such as:

1. What is cathodic protection?
2. How does it work?
3. How is it accomplished?
4. What can it do for you or, if you prefer, what can't it do for you?
5. How much does it cost and how much will it save?

Actually, we would be only kidding ourselves if we promised to provide definitive answers to the above questions. For, in fact, cathodic protection is more of an art than a science, with most of the solutions reached through use of empirical formulas which, in turn, are modified by trial and error applications in the field. Nevertheless, in this session we shall introduce the subject to you, trying to emphasize that cathodic protection is a valuable tool for combating corrosion in a variety of situations.

Let us begin with the definition. Cathodic protection is defined as the reduction or elimination of corrosion of a metal by making current flow to it from an electrolyte. Let us analyze the definition a bit and, hopefully, a better understanding will be realized. Perhaps this can be best accomplished by a brief look at the electrochemical theory of corrosion.

Corrosion Theory

Basically, there are only two types of corrosion, with many variations of these two types. One basic corrosion type is the galvanic or local cell corrosion, while the other is referred to as either electrolytic or stray-current corrosion.

Galvanic corrosion is a self-generated reaction resulting from differences in potentials when metal is placed in an electrolyte. Potential is the amount of energy which the metal possesses. Take steel for instance. Steel is not found naturally but is an alloy made from iron which occurs naturally as iron ore. Making of steel from iron ore requires vast amounts of energy. Although most of this energy is lost in the form of heat, some is stored in the steel as chemical energy and this is the potential of the alloy. In nature, energy tends to be lost or to run "downhill" to its natural condition. Corrosion is the result of the tendency of metals and alloys to revert to their natural state. And so, metals and alloys corrode with their corrosion products being quite similar to the natural ores from which they are made. An electrolyte, on the other hand, is any substance capable of conducting electricity. An ordering of metals listed according to their potentials in a particular electrolyte is referred to as a "Galvanic Series." The galvanic series for metal and alloys in seawater are shown below:

Galvanic Series of Some Commercial Metals and Alloys in Seawater

Active or Anodic

Magnesium
Magnesium Alloys
Zinc
Galvanized steel

Aluminum 1100

Aluminum 2024 (4.5 Cu, 1.5 Mg, 0.6 Mn)

Mild Steel
Wrought Iron
Cast Iron

13% Chromium Stainless Steel Type 410 (Active)
18-8 Stainless Steel Type 304 (Active)

Lead-Tin Solders
Lead
Tin

Muntz Metal
Manganese Bronze
Naval Brass

Galvanic Series of Some Commercial Metals and Alloys in Seawater (continued)

Active or Anodic

Nickel (Active)
76 Ni-16 Cr-7 Fe Alloy (Active)

60 Ni - 30 Mo - 6 Fe - 1 Mn

Yellow Brass
Admiralty Brass

Red Brass
Copper
Silicon Bronze

70:30 Cupro Nickel
G-Bronze
Silver Solder
Nickel (Passive)
76 Ni - 16 Cr - 7 Fe Alloy (Passive)

13% Chromium Stainless Steel Type 410 (Passive)
Titanium

18-8 Stainless Steel Type 304 (Passive)

Silver

Noble or cathodic

Graphite
Gold
Platinum

Dissimilar metals coupled together, as well as variations in the existing conditions on the surface of a single metal, can, and often do, result in galvanic corrosion. These variations could include nonhomogeneity of the metal, areas of good and poor coating integrity, stressed and unstressed areas, as well as local differences in the electrolyte. When two dissimilar metals are electrically interconnected and placed in a common electrolyte, the classic corrosion cell, the direct-current battery, is formed. Current is generated and one of the metals corrodes. Whether the current produces corrosion or the corrosion produces the current is analogous to the chicken-egg story and for our purposes is immaterial. The metal which corrodes is identified as the anode. The other metal receives protection and is referred to as the cathode. Current passes from the anode to the electrolyte and from the electrolyte to the cathode. The electrical connection between the anode and cathode completes the circuit. What determines which metal corrodes in a bimetallic couple is their relative positions on the galvanic series for that electrolyte. The metal higher or more active on the list, possessing more energy or potential, will be the anode, and corrode.

Electrolytic (stray-current) corrosion results from direct current from foreign sources such as a direct-current welder, electric railways, grounded direct-current electric power systems, electroplating plants, or cathodic protection systems entering and then leaving a metallic structure by way of the electrolyte. Again, where the current leaves the structure, we have an anode which corrodes.

The basic difference between the two types of corrosion is the source of the current: galvanic with self-generated currents, and electrolytic with external power source. You will note that the above discussion has been wholly restricted to direct current. One can readily see that with alternating current, the anode-cathode relationship would reverse as the polarity changed 60 times in a second, and thus reduced corrosion is realized. It has been estimated that 60-cycle alternating current causes only about 1 percent the damage produced by direct current of the same magnitude.

Basis for Cathodic Protection

From this brief review of the corrosion theory, it becomes rather obvious that to eliminate corrosion, the electric current which is producing the corrosion must be stopped. One method of doing this is by the use of coatings, the subject of the previous two discussions. Dielectric coatings reduce corrosion by impeding the flow of current. If a perfect coating were available and practical, one with infinite durability, excellent bond, 100 percent continuity, and having sufficient electrical resistance to prevent the flow of electric current, no corrosion would occur. Unfortunately, the perfect coating is awaiting development. And thus, cathodic protection is sometimes used to supplement coatings in the battle against corrosion.

Since we have found that electric current can cause corrosion and corrosion can generate electricity, why then cannot corrosion be prevented by the use of electric current? This was the thought of Sir Humphrey Davy, the English chemist, who is generally credited with developing the basic principle of cathodic protection and making its first application in 1824. The basic principle is to override, cancel, or negate the natural corrosion current with an opposing current from another source. When sufficient direct current is applied to the structure such that no differences in potential exist between formerly anodic and cathodic areas within the structure, corrosion is arrested. In essence, we are making all points on the surface on the structure a cathode by passing current to it and hence the title, cathodic protection. Cathodic protection thus is an active method of corrosion control, employing dynamic potentials to overcome the natural corrosion potentials. Coatings, on the other hand, are passive in that they merely impede the corrosion currents. Thus, you can appreciate that, in theory at least, the basis for cathodic protection is relatively simple, although as we will discover later, the practical designs for the various applications can differ substantially and become quite complex.

Cathodic Protection Methods

There are two methods by which corrosion can be controlled by cathodic protection, one being the galvanic or sacrificial anode method and the other being the impressed current method.

In the galvanic anode method, we control corrosion by purposely creating a corrosion cell through use of a bimetallic couple. The cell is designed in our favor through careful selection of the anode material so that it is more anodic than the structure to which it is coupled. In this manner, the current will pass from the galvanic anode which sacrifices itself and corrodes. The structure receives current and is protected. Corrosion is controlled, not stopped, as we have merely transferred the corrosion from our structure to the anode. Both the structure to be protected and the anode must be in a common electrolyte and be connected with an insulated electrical conductor. The insulation is required to prevent corrosion of the conductor and thus destruction of the metallic, current-return path.

Common sacrificial anodes used for protecting iron or steel are magnesium, zinc, and aluminum. Of these, magnesium is the most popular because of its greater driving voltage which makes it suitable for use in the higher resistivity electrolytes. Magnesium corrodes or is consumed at the rate of about 17 pounds per ampere-year, zinc at 26 pounds per ampere-year, and aluminum from 6 to 12 pounds per ampere-year.

The impressed current method of cathodic protection uses anodes which are energized by an external, direct-current power source. Since the anodes are not expected to provide the driving potential but are merely used to pass current from the external source on to the electrolyte, we can use more permanent-type materials for the anodes, those which are consumed very slowly. These include graphite, carbon, and high-silicon cast iron which are consumed at rates less than 2 pounds per ampere-year. The power source normally used is a rectifier which produces a direct current (actually rectified alternating current) from alternating current. Again, the anodes and the structure must be in a common electrolyte and connected together by way of the rectifier with an insulated conductor. Care must be exercised to connect the anodes and the structure to the correct terminals on the rectifier. The anodes are always connected to the positive terminal. If connected in reverse, accelerated corrosion of the structure will be experienced, although you will probably have the best-looking anodes in the country.

Each method of applying cathodic protection currents has characteristics which make it more applicable to particular situations than the other. Some of the more important of these characteristics are listed below:

Galvanic	Impressed Current
No external power required	External power required
Fixed-driving voltage	Variable voltage
Limited current output	Variable current output
Anodes consumed more rapidly	Anodes relatively long lived

With the fixed-driving voltage, the galvanic method is restricted to application in the lower resistivity electrolytes, but the possibility of interference on other structures is reduced. On the other hand, the impressed current method can be effectively used in the higher resistivity electrolytes but can also cause severe interference effects on other structures. Unavailability of alternating-current power may dictate the use of galvanic anodes. Therefore, the method selected is governed by the above conditions as well as other considerations and may turn out to be a compromise decision.

Applications and Limitations

Previously we have found that, theoretically, cathodic protection should be able to protect any metal in any electrolyte. Practically speaking though, for cathodic protection to be successful, we must have:

1. A suitable continuous electrolyte common to both the anode and the structure
2. A method of making current flow in the desired direction (to the structure)
3. A method of getting sufficient current to every point on the surface of the structure

Common suitable electrolytes include soil and water. Structures in air cannot be cathodically protected, although one can argue that the use of metallic zinc or metallic aluminum coatings on steel is a form of cathodic protection widely used in the atmosphere. Iron and steel are the most common materials protected cathodically, although more noble metals can be protected. Cathodic protection can be used on new structures and is often the most economical method of controlling corrosion on aged structures. Typical applications include protection of:

1. Exterior surfaces of buried tanks and pipelines
2. Interior surfaces of water tanks
3. Exterior surfaces of well casings
4. Tower footings
5. Watergates
6. Steel piling

One must remember that cathodic protection cannot plate metal on a surface; that is, it cannot replace metal lost to corrosion. In fact, in some reported cases, an aged pipeline will disclose a new leak soon after cathodic protection is applied. This is generally due to loosening of corrosion products which formerly were effectively sealing an existing hole.

An anode is limited to protecting only metal which it can "see"; the protective current cannot "reach through" a pipe to protect the interior since they are not in the same electrolyte. In other cases, one structure may shield another such that the protective current cannot get to the shielded structure.

Protecting the interior surfaces of pipelines, although possible, is normally not practical. However, this is sometimes done by some industries through use of special designs.

Bureau of Reclamation practice presently calls for employing cathodic protection when serious corrosion problems are indicated by previous experience or if a corrosion survey indicates an aggressive environment. In Bureau applications, cathodic protection is almost always considered for use with high-quality coatings. When used in conjunction with coatings, only the bare spots and holidays require protection, thus reducing the power requirement tremendously when compared to protecting the entire surface of uncoated metal or say 50 percent of a poorly coated structure. Power costs make cathodically protecting an uncoated or poorly coated structure extremely expensive, and thus, the application of a high-quality coating is normally justified except in special cases. Cathodic protection is most often applied to an existing structure when an anticipated corrosion problem develops long after construction.

Design

Due to the differences in the requirements for cathodic protection systems necessary to achieve protection, each system requires customized design. Rules are avoided simply because there are none applicable to every situation. The trick is to determine how much current is required and how to get it to every point on the surface of the structure. To obtain this information usually requires testing to ascertain:

1. The electrical continuity of the structure and its geometry
2. The resistance of the structure
3. The electrical characteristics of the electrolyte (resistivity)
4. The polarization characteristics of the structure

Even after the most thorough preliminary testing and careful design, each system requires checking and adjusting by trial and error after installation.

Criteria

The most commonly accepted criteria for protection of steel are the structure-to-electrolyte potentials of a negative 0.85 volt or more negative as referenced to a copper-copper sulfate electrode. This electrode consists of a copper rod immersed in a saturated copper sulfate solution. The solution is allowed to leach out through a porous plug to the electrolyte. This cell is sufficiently stable for most cathodic protection work. The potential is measured by means of either a high-resistance voltmeter or a potentiometer voltmeter.

When cathodic protection is used in conjunction with coatings, care should be exercised to maintain the protective potential within a range of from negative 0.85 to negative 1.50 volts. Overprotection can result in disbonding or stripping of some coatings through hydrogen evolution at the metal-coating interface.

Operation and Maintenance

Once cathodic protection systems have been designed, installed, and adjusted, they, of course, should be properly maintained for trouble-free service. Accurate and complete records should be maintained of the output currents of galvanic anodes and the output voltages and currents on impressed current systems. The record of the current outputs is required for galvanic anode systems to estimate anode life expectancy. Leads should be checked periodically to see that they, as well as their insulation, are intact. Rectifiers for impressed current systems should be kept free of debris, and their output should never exceed their rated capacities in either voltage or current. Any large changes either in current or voltage indicate need for further investigation. Also, about once a year, a potential survey should be conducted to ascertain if adequate protection is being obtained.

Cost and Savings

At the beginning of this period, I indicated that we would discuss cathodic protection costs and savings. Unfortunately, again no general rule applies. However, perhaps we can discuss some parameters which affect the cost of cathodic protection:

1. Size and shape of structure to be protected. - It is, of course, obvious that, other considerations being equal, a small structure would be cheaper to protect than a large structure and that simple shapes are more easily protected than complex shapes.

2. Electrolyte characteristics. - Less power is required in the lower resistivity media, and thus, almost always ground (anode) beds are installed in the lower resistivity soils.

3. Type and integrity of coating. - Protecting surfaces coated with highly insulative coatings such as coal-tar enamel would be less costly than protecting surfaces coated with cement mortar. The coal-tar-enamel-coated structure requires protection only at the pinholes and bare spots, as contrasted with the highly conductive (when wet or damp) cement-mortar-coated structure consuming current over its entire surface.

4. The cost of available power would, of course, be an important consideration.

Review and study of some of the literature and reports indicate that the range in costs for installation of pipeline cathodic protection systems varies considerably from a low of $0.01 per square foot to as much as $0.20 per square foot, with the average being perhaps $0.07 per square foot. This at least should give you an idea of the order of magnitude of cathodic protection costs.

The savings can best be illustrated by examples. The initial cost for installing the cathodic protection system for the Panama Canal gates reportedly was less than one-half of 1 percent of the cost of gate replacement. One also hears stories in which one leak in an oil line can eat up the entire year's profits.

Closer to home, water, of course, is a very valuable commodity, particularly to farmers who need one last watering to mature their crops. What kind of savings do you think they would assign to corrosion prevention? Particularly if a hole develops and it turns out to be their share which is going down the drain or, worse yet, flooding a ready-to-harvest orchard. I have the feeling they would assign a rather high figure, don't you?

Although one could probably assign astronomical values to corrosion costs, to be meaningful, the cost should be determined using a sound basis. Some of the costs attributable to corrosion are shown below.

The direct costs are determined rather straightforwardly:

1. Cost of product lost through leaks
2. Loss of revenue
3. Cost of labor and materials to repair the leak
4. Damage to landowners for land and excavation
5. Cost of overhead, engineering, and testing in leak repair
6. Increase in annual depreciation due to shortened life

On the other hand, indirect costs are difficult to assess. This is true due to their intangible nature. What value do we place on loss of life or limb? Or the psychological distress attendant to unpredictable, catastrophic failures? These are the things to be considered in assessing the indirect costs of corrosion.

From the above discussion, one can see that assigning costs and savings to corrosion control can become quite complex. But in this day and age when emphasis is placed on economic justification, you must be armed with a sound cost and savings estimate which considers the above as well as other factors in presenting a proposal to management. The days are gone when you could merely say: "I think it would be well worth the investment."

Summary

To summarize briefly, we have found that cathodic protection is a highly adaptable and effective means of corrosion control. Although the theory is relatively simple, the practical applications are usually quite complex, and so this method of combating corrosion cannot be applied indiscriminately or without careful investigation. However, when properly applied, cathodic protection can be one of the more economical corrosion control methods, and as such, you should keep it in mind for augmenting coatings in the never-ending battle against corrosion.

Water Systems Operation and Maintenance Workshop - 1990

CORROSION CONTROL BY COATINGS AND CATHODIC PROTECTION - PART 2

Session 11-2

John Baker

Rising costs dictate an active corrosion control program for water control structures. Prevention of corrosion may take several forms, such as correct selection of the base materials, application of cathodic protection to metals, etc. However, protective coatings often emerge as the most economical means of mitigating corrosion. Coatings for new construction are specified with the objective of maximum durability and economy. Ideally, these coatings would last the life of the structure. Unfortunately, this goal rarely is attained. Coatings nearly always require maintenance and perhaps replacement. Maintenance painting then proceeds with the problems which attend new construction plus a number of additional handicaps in many cases.

The Bureau of Reclamation maintains a coatings and corrosion laboratory with the functions of investigating, selecting, specifying, and testing coatings for Bureau structures, and consulting with and assisting field offices regarding all aspects of their use. These Bureau facilities are accessible to water districts through normal Bureau channels. We welcome the opportunity to lend assistance with maintenance problems whenever possible.

Before discussing the selection of coatings, the principal topic of this session, some general comments on painting may be in order.

First, surfaces are painted primarily for two reasons: (1) for decorative purposes, and (2) for protection of the structure. The former applies mainly to surfaces in view and is usually secondary in importance to protection, although appearance is assuming greater prominence in these days of environmental concern. In either case, the economic objective is to minimize the cost of painting. Consideration of cost factors will reveal that durability of the coating strongly influences painting cost when this is viewed on an annual basis, dollar per square foot per year of service. Thus, a more expensive paint and/or one requiring more meticulous adherence to proper application techniques may well be justified if they enhance durability significantly. Precluding frequent maintenance may pay large dividends in the long run.

What then are the essentials of providing effective coating protection? The critical steps are:

1. The correct selection of the paint material for the exposure
2. Quality control of the paint supplied
3. Correct application

The first of these is covered in this session and the third will be discussed in another session. The importance of the second, quality control, may be realized from the fact that rejection has occurred at the rate of 20 to 25 percent of samples tested by the Bureau. Failure to meet the established quality standards by one out of four to five materials submitted for use on Bureau structures points up a very real danger that originally defective materials may lower the durability of coatings. The Bureau considers that testing is warranted for all sizable quantities of paint.

Each of these steps must be executed correctly!

The Coating Selection Process

Returning to the main topic of this discussion, step 1 (correct selection of the type of material), it may be well to elaborate on some features of the selection process. Obviously, this is a process of matching material characteristics to the requirements of the situation. Prominent among these is resistance to the exposure in which the coating will be placed. One must learn which elements of the exposure are most likely to cause deterioration of a coating and gain some idea of their intensity. Some exposures are listed below together with typical deteriorative conditions often present.

Basic categories of exposures	Some deteriorative conditions possibly present in the exposure
1. Immersion	Water, chemicals (low concentrations), erosion, abrasion, cavitation, high water velocities, corrosion currents, biological organisms
2. Burial	Water, chemicals, abrasion, soil stress and backfill damage, corrosion currents, biological organisms
3. Alternate submergence	Water, chemicals (low concentrations), erosion, abrasion, cavitation, high water velocities, corrosion currents, biological organisms, sunlight, air, intermittent water, temperature extremes
4. Atmospheric (indoor and outdoor)	Sunlight, air, intermittent water, abrasion, temperature extremes
5. Chemical	Chemicals (high concentrations), water, erosion, abrasion

Of course, the nature of the substrate and its surface condition should be known. The substrate may be wood, masonry, iron or steel, nonferrous metal, or a variety of other materials. Its condition may be new or old, sound or deteriorated, painted or unpainted, and various surface conditions can exist which may affect the choice of coatings.

Other circumstances enter the picture. For example, can the coating be applied under the existing conditions? Will the work be done by experienced painters or by less-skilled personnel? Will it be done under contract? What durability of protection is sought? Are there rigid limits on cost? What is the urgency of performing the work? What is known about the past performance of various coatings exposed to the same or similar environments in the vicinity?

How do we evaluate coating characteristics? Briefly, this is based on some or all of the following sources: manufacturers' data, independent service histories from our own or outside sources, and our laboratory and field testing. Length of recorded trouble-free service, resistance to various exposure conditions, surface preparation and application standards and latitude to tolerate substandard conditions, ease of application under field conditions, and cost are considered.

With the foregoing information in hand, the various features of available coatings can be matched to the exposure requirements and other circumstances. This process, however obvious, has been recited because it underlines the need for critical analysis of the exposure, particularly when a special problem exists, and the desirability of obtaining well-rounded data on coating characteristics of potential materials for that exposure. Experience dictates that some aspects of painting situations often are overlooked.

Before discussing specific coatings, we should recognize the difficulties added to the selection problem by maintenance circumstances. First, there may be an existing coating, and the question may be whether to save it or replace it. In either case, the old coating requires special consideration. Next, painting in place may pose problems of access and ventilation. In a few instances, drying up the area sufficiently may be very difficult. Finally, the coating may have to be applied within strict time limitations, or the job postponed until sufficient time can be programed. The overall painting plan must cope with these obstacles.

As an aid to planning coating maintenance, we recommend regular inspection of coatings on a projectwide basis and records kept to provide histories on the coatings. Such records are useful, not only as a ready reference as to the condition of a coating, but in forecasting the time when maintenance or replacement will be required. This is summarized in current Designers' Operating Criteria in which are given suggested inspection schedules for various types of paints. Also, a particular problem sometimes arises during inspections in arriving at a correct evaluation of coating condition, interpreting defects, and deciding what maintenance or replacement is required. This is discussed under "Maintenance Painting" in the Paint Manual.

Specific Coatings

Current Bureau coating selections, based on many years of experience with water control structures, are best summarized in painting guide paragraphs C-1000 and C-1001 which cover most of the common metalwork and exposures. The Bureau Paint Manual provides much broader background information on Bureau coatings than can be included here and is strongly recommended for those with responsibility for coating selection and application. Again, water districts may consult Bureau coating specialists on any problem they encounter, current coating selections, or for available information on specific materials.

Some structures are more critical than others. For example, we usually are more concerned with the piping and valves carrying water than with the exterior of the valve house. Thus, the discussion of specific coating selections will emphasize painting metal in the more vital exposures.

Table 1 summarizes some selection factors associated with selected coatings. As with any summary, it is incomplete and therefore can be misleading if it is used as the sole basis for coating selection. The following discussion on the properties of selected coatings will provide a more reliable basis for coating selection. Final selection of any coating should be based on guide paragraph C-1001, experience, or reliable consultation.

Metalwork in freshwater immersion, alternate submergence, and buried exposures. - Coal-tar enamel and cement-mortar are the only two coatings presently considered to be "virtually permanent" and which may last the life of the structure under favorable circumstances. Accordingly, they are specified for the interior and exterior of piping whenever possible. The long life of coal-tar enamel justifies its admittedly obnoxious application characteristics. Enamel is almost invariably applied under contract since unusual equipment and specialized skills are required for its application. Coal-tar enamel cannot now be used for potable water applications.

Cement-mortar is most often shop applied for new construction. However, in-place mortar lining is highly applicable to water piping in that leaking, badly deteriorated steel lines 4 inches in diameter and over can be restored to full serviceability by this process. Mortar tolerates both moisture and surface preparation so poorly as to be unthinkable for other coatings. It also plugs up leaks effectively. While in-place mortar lining is expensive, it usually costs much less than replacing the piping.

Enamel and mortar are the two principal coatings for exterior, buried piping. Enamel is an excellent electrical insulator and, therefore, cuts cathodic protection costs to a minimum since only the few small, bare areas actually require the protective current. Enamel with its protective wraps requires care during installation to prevent backfill damage, while cement-mortar is highly resistant to mechanical damage.

Health and safety considerations are causing coal-tar enamel to be used less and less frequently because they are constantly decreasing the number of qualified applicators.

Unfortunately, the limitations on both coatings prevent broader usage. Neither can be applied to complex shapes. Enamel is vulnerable to wide temperature extremes and sunlight degradation. Mortar is considered not to resist water velocities over about 20 feet per second, and wetting and drying and freeze-thaw exposures are avoided.

Coatings with a normal life of about 20 years are specified for complex shapes and exposures involving temperature extremes and/or sunlight, such as aboveground piping which will be empty in winter, and bulkhead and radial gates.

Coal-tar epoxy paint represents a new class of industrial level paints which can be expected to appear more and more frequently in our work. These materials usually feature two or more components which react chemically to set or cure the coating. Limited pot life and temperature sensitivity complicate the application. These disadvantages must be outweighed by expectation of superior durability.

Coal-tar epoxy paint replaces CA-50 in current Bureau practice, and we expect two to three times the durability on metalwork in immersion service. The material degrades slowly in sunlight so it is specified only for gates, trashracks, and piping which will be out of direct light. Also, it may not be used for potable water applications.

VR-3 and VR-6 vinyl resin systems have performed well for many years in immersion and alternate submergence exposures such as water tank interiors, radial gates, and water piping. The thicker VR-6 coating better resists abrasion from floating debris on radial gates and trashracks. The vinyls now largely replace phenolic red lead and aluminum systems formerly specified but which performed poorly in several instances. If VR-3 and VR-6 vinyl resin systems are to be in contact with potable water, the raw materials and final product must conform to EPA (Environmental Protection Agency) regulations.

Care should be taken to avoid applying VR-3 to surfaces formerly coated with coal-tar base paints. Poor adhesion may result even though especially rigorous sandblast cleaning is performed. Application of a VR-6 primer coat prior to the VR-3 coating system will remedy the bond deficiency or, better yet, another coal-tar base coating may be selected.

Hot-dip galvanizing, while usually excellent in atmospheric exposures and sometimes in immersion, fails rapidly in certain waters. Unfortunately, predicting which waters will be aggressive is uncertain even when one has gone to the trouble of having the water analyzed. VR-3 is preferred because it resists most waters well.

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Table 1. - Approximation of some selection factors for protective coatings

	Exposure										
Coating	Water	Water Vel.	Cavitation	Erosion	Abrasion	Soil	Weathering	Temp.	Surf. Prep.	Application	Specification
Cemt.-Mort.											
Lining	A+	B	C	B	C	-	-	B	Clean	Spec.	AWWA C205+Mod.
Coating	A+	-	-	-	C	A+	-	B	Clean	Spec.	AWWA C205+Mod.
CT Enamel											
Lining	A+	A	C	C	A	-	-	B	SB	Spec.	AWWA C203+Mod.
Coating	A+	-	-	-	A	A	-	B	SB	Spec.	AWWA C203+Mod.
(PRITEC)	A+	-	-	-	A(?)	A+	B(?)	B	SB	PLNT	Bureau Specs M58
CLD-Tapes	A(?)	-	-	-	-	A(?)	-	-	SB	Spec.	AWWA C214
CT Epoxy	A+	A	C	C	B	?	C	B	SB	---	MIL-P-23236, Type 1, Class 2
Epoxy	A(?)	A(?)	C	B(?)	B(?)	?	C	B	SB	---	MIL-C-4556
Fus. Epoxy	A(?)	A(?)	-	B(?)	A	A(?)	B	B	SB	Spec.	AWWA (?)
Vinyl	B	-	C	D	C	-	A	B	SB	---	VR-3
Vinyl	A	-	C	B	B	-	A	B	SB	---	VR-6
Alum. Epoxy	A(?)	-	-	-	-	-	A(?)	-	SB	---	None
Sil. Alk. En.	-	-	-	-	-	-	A	-	Primed	---	TT-E-490E (Others)
Acrylic	-	-	-	-	-	-	A	-	Primed	---	TT-P-19C (Others)
Elastomeric	A(?)	A(?)	A(?)	A(?)	A(?)	-	B(?)	-	SB	---	None

Note: This table is INCOMPLETE and does not include all selection factors. It can be MISLEADING because of this. It was prepared for lecture purposes and is not to be construed as representing an overall Bureau evaluation.

The Bureau frequently permits the galvanizing of small gates which can be removed easily for rehabilitation. Here, VR-3 would be preferred, but the better resistance of galvanized gates to handling, shipping, and installation offers considerable advantage. The question is often raised about the comparative costs of galvanizing to that of the red lead aluminum or VR-3 vinyl paint systems for slide gates. The Armco Company has furnished us with these costs: for 12- and 24-inch model 101 gates, painting costs from one-third more to twice that for galvanizing; the 30- and 36-inch gates cost approximately the same for either of these protective coatings; and for gates over 36 inches, the cost for galvanizing rises sharply over that of painting and considerable difficulty has been experienced with warping of steel members of the larger gates during the hot-dip galvanizing. Warping has also been experienced in galvanizing radial gates.

Erosion is an important factor in the destruction of some linings in siphons fed by open canals. Coal-tar enamel in invert areas has been seriously damaged in such locations after 5 or 6 years of service. Cement-mortar is also damaged by this type of erosion but, being a thicker coating, the effects may not be seriously detrimental as soon as in the case of enamel. Laboratory and field studies have shown neoprene to offer excellent resistance to this erosive environment; however, the difficulty of application and high costs limit the use of neoprene to special cases. Vinyl resin paint is generally specified for the interior surfaces of exposed steel siphons. By adding sand (20- to 30-mesh size) to the third coat of VR-3 vinyl resin paint in the invert, the erosion resistance of the paint is increased appreciably. This method of combating erosion appears to be the most economical approach.

Metal in atmospheric exposures. - The lesser demands on protective coatings in atmospheric exposures and their greater accessibility for recoating justify a lower and less costly level of protection than is employed for exposures discussed earlier. Accordingly, cheaper wire brush cleaning is followed by iron oxide primer, TT-P-636, and enamel or aluminum finish paints to provide protection and the desired appearance. For galvanized surfaces, cleaning is minimal and a zinc dust-zinc oxide primer provides the needed adhesion without special surface pretreatments.

In recent years, the requirement that structures present an attractive appearance has prompted extensive use of gloss and semigloss enamels in many colors. It is recognized that this involves some sacrifice in durability over that from aluminum paints so long specified.

Recently, silicone-alkyd enamels meeting TT-E-1593B (gloss) and TT-E-490E (semigloss) have been adopted for metalwork in exterior exposures with the expectation for longer service than is being obtained with alkyd enamels TT-E-489G (gloss) and TT-E-529E (semigloss). Note: The letter following the specifications number designates the most recent modification(s) of the specifications. Modifications of specifications may or may not be significant.

Painting concrete and masonry. - In general, the Bureau avoids painting concrete on the theory that good concrete needs no protection. However, below-grade concrete in buildings is dampproofed, freeze-thaw damage afflicts some older structures, and occasionally underwater concrete requires sealing. Further, various concrete and masonry units require sealing and/or beautification treatments.

The choice of coating should take into account the presence and location of water, if any. Surfaces to be sealed against water should be coated on the water side, even if the water is only under low pressure. Thus, sealing of a dam with a neoprene paint would be done on the upstream face (including joints). If painting of the downstream face were required, say for decorative purposes or to shed rainwater, a breather-type paint such as latex or portland cement paint would be indicated. Water vapor from the interior could then escape without building up pressures which would blister a membrane-type paint. Portland cement paints must be used with caution since they are very difficult to recoat with other types of coatings.

Freeze-thaw damage to concrete parapets on dams and elsewhere has long been treated by the old Oregon State Highway method of applying linseed oil and paint to seal against rainwater. This treatment produced good results; however, the State of Oregon now uses a much simpler latex paint for this purpose. Also, Bureau tests now in progress may yield a synthetic paint offering superior protection. On an interim basis, we are recommending one of the materials in test, an epoxy paint applied as a thinned, clear, priming coat followed by the same paint, unthinned, and in a color matching the concrete.

General

In the maintenance of existing submerged or buried steel structures, repainting may be very difficult or impossible. In this case, one method of affording continued protection may be to apply cathodic protection which often can arrest corrosion in areas where the coating is defective. If this is contemplated, one should be aware that cathodic protection currents develop an alkaline environment at the metal surface which can deteriorate the bond of certain coatings. The phenolic-resin coatings, type IV red lead primer, and phenolic aluminum have been so affected. However, coal-tar enamel, coal-tar epoxy, and VR-3 and VR-6 vinyl resin coatings are "compatible" with cathodic protection. A second point to bear in mind is that cathodic protection installations must be adjusted and monitored to maintain the correct potentials and currents. Otherwise, excessive protection may damage or destroy the coating. Protection potentials should not be more negative than -1.5 volts as referenced to a copper-copper sulfate electrode. A coating is, of course, highly desirable in conjunction with cathodic protection since the coating reduces current demand on the cathodic protection system, as well as affording some measure of protection to the metal.

Important note should be taken of the influence safety and environmental concerns, as expressed in the OSHA (Occupational Safety and Health Act) regulations, are exerting on the protective coating field. Restrictions are tightening on blast cleaning in preparation for painting, on painting procedures, and on the character of coatings. It is as yet too early to predict specifically the end results of OSHA. However, we can safely expect that a new generation of "pollution-free" coatings will evolve gradually and will replace presently objectionable materials. The conversion is bound to cause some dislocations and inconvenience, and the new materials initially will not be backed by the long service histories of present paints. They probably will cost more. However, as in the latex house paints now replacing oil-base paints, some clear improvements and advantages can also be anticipated.

New coatings we have looked at recently include highly elastomeric, two-component urethane systems which go on concrete or steel. They have good immersion, abrasion, and erosion resistance but may have only fair weathering resistance. They have been applied in field locations. An "underwater" epoxy paint could actually be applied under water with acceptable ease and cured quickly at about 55 °F. However, the principal use foreseen for it would be in cool, very humid, damp, or wet places where conventional materials simply could not be used. Samples of these urethane and epoxy paints are in various tests to be inspected later; they seem particularly well suited to maintenance painting circumstances. We also have samples of acrylic emulsion (water-base) paints which appear to be the coming answer to OSHA for metalwork in atmospheric exposure, especially electrical equipment. Water-base epoxy paints for immersion exposures are being developed by industry but are not here yet.

Environmental and OSHA factors are speeding up the Bureau's coating programs. Along with other organizations, both public and private, it will be necessary for the Bureau to make effective use of OPE (other people's experience). Communications between the E&R (Engineering and Research) Center and other organizations, both Bureau and non-Bureau connected, are more vital than ever before.

Among the items of immediate concern are:

1. Elimination of TT-P-86G, type IV, red lead primer from Bureau specifications. At present, two alternative coatings are available:

   a. Cook Paint and Varnish Company's stock No. 391-N-391 Corobond No. 167 lead- and chromate-free primer

   b. Valspar Corporation's No. 13-R-28 Chromax primer

   The number of potential suppliers will be expanded when a new specification for lead- and chromate-free primers is developed or adopted. These primers will also be suitable for use in applications which now use TT-P-636D alkyd oxide-chromate primer.

2. The State of California is currently planning to enforce air pollution control regulations which will effectively eliminate VR-3 and VR-6 vinyl coating systems in the state by September 1987. Among the possible replacements which are already scheduled to be used on some Bureau projects are a high-solids two-package epoxy which can be used by itself or with an aliphatic polyurethane topcoat, if appearance is important. Conventional epoxies, metallizing, 100-percent-solids epoxies, and elastomeric aliphatic polyurethanes are among the other types of coatings which are being considered.

Summary

The Bureau attempts initially to provide the most durable protective coatings currently available for the several kinds of surfaces on water control structures. Nonetheless, maintenance of these coatings eventually becomes necessary. Correct coating selection, quality materials, and meticulous field application of maintenance coatings contribute to effective, long-lived protection at minimum cost over the long pull.

This discussion focused on the basic factors in the coating selection process; it does not pretend to have covered all of them. Likewise, description of the most common materials did not include all materials nor all of their pertinent attributes. Moreover, many coating situations present seemingly unique combinations of problems which cannot be covered herein.

Those responsible for coating selections are again referred to the guide paragraphs which spell out the details of current Bureau practice and to the Paint Manual for broad background information on particular coatings and all phases of the painting operation. They may also contact Bureau coating specialists for discussion of particular problems on their projects.

Table 2. - Commonly used Bureau coatings and their possible replacement or alternative coatings

Item	Present coating column 1	Advantages column 2	Disadvantages column 3	Repl. or alt. coatings column 4	Advantages column 5	Disadvantages column 6
1	Coal-tar enamel	Proven exceptionally long, useful life span.	Acute health and safety problems and must be hot applied. Specialized application technique required.	Coal-tar epoxy.	Fewer acute health and safety problems, cold applied, although 2-package materials.	Relatively long curing times, mandatory short maximum recoating times, cannot be applied at low temperatures (below 50 °F).
2	Coal-tar epoxy	Bureau has had reasonably satisfactory experience with this coating.	See column 6, item 1.	Hi-solids, 100% solids, or faster curing epoxies.	Shorter downtimes, and longer critical recoating times with many of these types. No solvent problems with 100% types.	Lack of extensive Bureau experience with these types of coatings. Special application equipment may be required. Low temperature limitations.
3	Coal-tar enamel and Coal-tar epoxy	See Column 2, items 1 and 2.	See column 3, items 1 and 2.	Elastomeric coatings, hi-solids and 100% solids.	Same as column 5, items 1 and 2, plus high dry film thicknesses and good abrasion and erosion resistance. No solvent problems with 100% types. Aliphatic polyurethane types weather well.	Same as column 6, item 2, plus possible potential of "sheeting" off if adhesion is lost. Aromatic polyurethane types do not weather well.

Table 2. - Commonly used Bureau coatings and their possible replacement or alternative coatings - Continued

Item	Present coating column 1	Advantages column 2	Disadvantages column 3	Repl. or alt. coatings column 4	Advantages column 5	Disadvantages column 6
4	VR-3 and VR-6 Vinyls	Equally suitable for immersion or atmospheric exposure, long useful life span with good surface preparation and application. May be applied under relatively low temperature conditions.	Multiple coats required (labor intensive), sensitive to surface preparation and application conditions.	Conventional epoxies.	Fewer coats required (less labor intensive), slightly less sensitive to surface preparation and application conditions, and shorter downtimes.	Chalk when exposed to sunlight, and cannot be applied at low temperatures (below approximately 50 °F - 55 °F).
5	VR-3 and VR-6 Vinyls	See column 2, item 4.	See column 3, item 4.	Hi-solids, 100% solids, or faster curing epoxies.	Same as column 5, item 2, plus appreciably shorter application and downtimes. May be topcoated with aliphatic polyurethane to prevent chalking and improve long-term appearance and durability. Topcoating will reduce a portion of the savings in application and downtimes.	Same as column 6, item 2 for these coatings and conventional epoxies.

Table 2. - Commonly used Bureau coatings and their possible replacement or alternative coatings - Continued

Item	Present coating column 1	Advantages column 2	Disadvantages column 3	Repl. or alt. coatings column 4	Advantages column 5	Disadvantages column 6
6	VR-3 and VR-6 Vinyls	See column 2, item 4.	See column 3, item 4.	Elastomeric coatings, Hi-solids and 100% solids.	Same as column 5, item 3, for these coatings plus shorter downtimes.	Same as column 6, item 3, above these for coatings.
7	Coal-tar enamel Coal-tar epoxy VR-3 and VR-6 Vinyls	See column 2, items 1, 2, and 4.	See column 3, items 1, 2, and 4.	Metallizing	Very short downtime without an organic topcoat, relatively short downtime with an organic topcoat. Expected long useful life span.	Surface preparation and application very critical, relatively high initial cost. Requires experienced applicators.
8	1/ TT-P-86G, type IV, Red Lead Primer	Proven, usually long, useful life span. Suitable for applications where there is heavy condensation.	Acute health and safety problems.	2/ Lead and chromate free anticorrosive primers.	No acute health and safety problems, some types can be used with a relatively broad spectrum of topcoats. It is almost a "universal" alkyd or oil-based primer for steel.	Use of these primers on Bureau features as alternative coatings has begun so recently that no long-term data are available able.

Table 2. - Commonly used Bureau coatings and their possible replacement or alternative coatings - Continued

Item	Present coating column 1	Advantages column 2	Disadvantages column 3	Repl. or alt. coatings column 4	Advantages column 5	Disadvantages column 6
9	TT-P-636D Alkyd oxide - chromate primer	Proven satisfactory useful life span, widely available from coatings suppliers.	Contains chromate which is sometimes "under fire" for health and environmental reasons, not suitable for applications where there is heavy condensations.	2/ Lead and chromate free anticorrosive primers.	See column 5, item 8.	See column 6, item 8.
10	TT-E-490E Silicone alkyd semi-gloss enamel	Good weathering properties, easy field application.	Solvent-borne coating, may streak if there is overflow from a water tank.	3/ Acrylic emulsion	Good weathering properties and ultraviolet resistance, water-borne coating.	Cannot be applied at low (below approximately 40 °F) temperatures. Limited Bureau experience with these coatings on metal.

1/ TT-P-86G, type IV, Read Lead Primer is in the process of being phased out of Bureau specifications for health and safety reasons. Removal of old red lead coatings is posing an expensive problem in some areas, at present, because of safety and environmental considerations. Presently available alternatives for TT-P-86G, type IV, Red Lead Primer are discussed in footnote 2/.

2/ The Materials Science Section, D-1521, of the E&R Center is currently completing a testing program which will result in the deletion of TT-P-86G, type IV, Red Lead Primer from Bureau specifications. The two alternative coatings which are currently available are Coroband No. 167, stock No. 391-N-167, produced by the Cook Paint and Varnish Company, and Chromox Primer 13-R-28, produced by the Valspar Corporation. These primers are lead and chromate free. Their performance has been equal to or better than that of TT-P-86G, type IV, Red Lead Primer in untopcoated water immersion tests. When testing has been completed, a new coating specification for lead and chromate free primer will be written or adopted. Because this primer will also be an alternative for TT-P-636D Alkyd Oxide - Chromate Primer, economics will be able to determine whether this primer becomes our "universal" alkyd or oil-based primer for steel. Also, we will be completely prepared if the use of chromates in coatings is banned in the future.

3/ There are new acrylic emulsion coatings which have shown promise as coatings for architectural steel. The Bureau is quite interested in these coatings, particularly if water-borne primers can be used with these coatings to form a completely water-borne system. The U.S. Navy is currently investigating the possibility of replacing silicone alkyd coatings with acrylic emulsion coatings on the superstructures of naval vessels. A planned investigation of acrylic emulsion versus silicone alkyd coating systems is on the research agenda of the Materials Science Section of the E&R Center.

Water Systems Operation and Maintenance Workshop - 1990

CORROSION CONTROL BY COATINGS AND CATHODIC PROTECTION - PART 3

Session 11-3

Clair Haverland

This session is concerned with on-the-job application of coatings. In this connection, the single most important idea to be retained from this discussion is that obtaining the expected service from a coating system depends directly on faithfully performing each phase of the application in accordance with the prescribed standards. This generalization applies to all paints, all painting conditions, and irrespective of the material or item being painted. The detailed requirements themselves will vary with circumstances, but awareness that standards exist and must be met will often provide the incentive needed to get an application off to a good start. It should lead to inquiry as to what standards apply, and when these are made known to the painters, should encourage substantially sounder coatings. In turn, correctly selected and applied coatings, which serve longer and require less maintenance, contribute importantly to reduction of coating cost on a dollar per square foot per year basis, which is the ultimate objective.

The coating maintenance problems in water operations often differ from the coating application problems in new construction, primarily in that the item to be painted is ordinarily in place; may have corroded severely; and may have at least the remnant of an old coating, one which may require maintenance or replacement under adverse circumstances. In new construction, new items are usually painted under much more favorable circumstances. It must be borne in mind that, although the circumstances may complicate an application, the same relationship applies between the service obtained and the soundness of the initial application operation.

A coating does not try a little harder just because the application was difficult to perform. More often it fails prematurely because the application standards were not adhered to. The problem, then, is to create at least adequate conditions for paint application. This will often require considerable ingenuity plus advance planning to secure the most favorable seasonal and other conditions.

Advance planning for all contingencies may be the most important aspect of many applications.

A painting operation, whether to concrete, to wood, or to metal and irrespective of the paint involved, can usually be divided into three parts:

1. Preparation of the base surface
2. Mixing or other preparation of the paint
3. Application to the surface

This discussion will consider these phases in the light of maintenance conditions and primarily with respect to metal as the most important material needing protection. The application characteristics of Bureau of Reclamation applied paints, as well as some recently developed paints, will be reviewed.

Before discussing the three phases of an application, some special problems likely to be encountered in maintenance circumstances should be considered. Winter painting introduces the possibility that part or all of the work may proceed below acceptable temperatures (45 °F is the usual Bureau minimum). This may often be countered by moving portable items under cover. Immovable aboveground items can usually be coated at acceptable temperatures if the work is scheduled (and advance preparations completed) for early fall or late spring.

The actual painting (especially the first coat) should be planned so as to start during the warmest part of the day; metal exposed to the sun may get as much as 20 °F warmer than the surrounding air. Underground metalwork will seldom be as low as 45 °F. However, in many such locations, the air can be heated to increase the drying rate.

The plans for the application should include extended drying time when temperatures for the curing can be expected to average substantially below normal (say 70 °F). It is sometimes also possible to select a fast-drying paint like VR-3 as being preferable in a particular situation to a material such as CA-50.

In some cases, other aspects of painting conditions can create problems, for instance, ventilation and humidity control. Positive movement of air is sometimes almost mandatory in underground painting to exhaust paint fumes, for drying, and to keep the area habitable to painters. Bulkheading to isolate an area and fans and vents placed in the bulkheads to control the air may be indicated. Underground metalwork sometimes sweats under these conditions and this must be prevented or fresh sandblasting and newly applied paint may be affected. Sweating (moisture condensation) occurs because the dewpoint is reached (even though only at the surface of the metal). Drawing moist, relatively warm air through a cold pipe is the most common cause. Remedying the situation is usually a matter of experimentation; however, heating the air and slowing its movement will often help. Operations may have to be suspended and the work area sealed up during rainy weather. Sources of moisture along the path of incoming air should be avoided, such as ponded water or moist areas at the lower levels of a structure.

When painting aboveground metal, it will usually be necessary to coat all areas during the same day as they were cleaned to counter the effects of nighttime condensation.

The more stringent enforcement of safety practices, together with a few new prohibitions under the OSHA (Occupational Safety and Health Act), likely will complicate painting, especially in enclosed areas. New chemically reactive paints pose unfamiliar hazards; cautions in manufacturers' literature must be read and carried out carefully. New knowledge developed about solvents indicates in a few cases that subtle, extremely serious ailments may materialize from inhalation or skin contact, sometimes in a delayed and cumulative manner. If air-feed or other respirators, skin creams, or plastic gloves are recommended, they should be obtained and their use enforced.

The above are a few typical and common obstacles to painting under maintenance conditions. There are others, and many times several combine to present a particularly knotty problem. The desirability of attempting to anticipate painting conditions is stressed because failure to do so will lessen the soundness of the paint film applied and hence the economy of corrosion protection.

Surface Preparation

Reference to Guide Paragraph C-126, Cleaning and Painting of Metalwork, will show that cleaning methods, called A, B, and C, are the ones primarily used in new construction. Others find limited use in special circumstances.

Method A requires merely the removal of foreign substances from an existing coating. In maintenance, this method would apply when an aged coating requires one or two topcoats to extend its service life. Scum and dirt are usually the materials to be removed, and vigorous scrubbing with stiff brushes and a detergent soap solution (followed by thorough rinsing) may suffice. Other contaminants, if present, must also be removed by appropriate means since any barrier to proper adhesion of the topcoat will reduce the value of the work performed.

Method B cleaning is intended as preparation for paints to be in atmospheric exposure, and specifically not for those in underwater exposure. Briefly, effective method B cleaning removes all loose materials on a surface, usually by wire brushing, chipping, or scraping. In new construction, it is assumed that no coating is present. This may not be true in maintenance work, and if the previous paint has failed in adhesion (peeling, flaking, lifting), a cleaning method such as scraping may be preferred to wire brushing which would leave behind large areas of poorly bonded old paint; the latter might soon cause premature failure of the repaired coating.

High-pressure water blasting may meet method B standards, and is discussed later. Of course, blast cleaning is permitted under method B and would produce better and faster results.

Method C, blast cleaning, is intended to remove all obstacles to the best chemical and physical adhesion of a coating. It is required cleaning for all underwater coatings because no coating we know of will dependably give long service in this exposure without the most thorough surface preparation.

We believe that a small sandblast unit would be a valuable addition to the equipment of a water district. At an equipment cost of perhaps $1,500 (exclusive of compressor), the district not only can perform sandblast cleaning where it is essential, but can reduce the cost and increase the quality of other applications. Light blasting is considered the most rapid, hence most economical cleaning method meeting (and exceeding) method B standards. Further thorough cleaning of cracks and grooves in concrete canal linings is essential before placing joint filler material if good service is to be obtained, and sandblasting is generally the most practical and economical method of accomplishing this where sizable areas are involved. Blasting, similarly, is best for cleaning concrete of old coatings, particularly curing compound, over which coatings will not perform satisfactorily. The many uses of a small commercial sandblast unit thus warrant its serious consideration where coating maintenance is a continuing problem.

A problem arising repeatedly in maintenance work occurs when a coating in submergence exposure must be repaired or replaced, but because of the physical circumstances, sandblasting is clearly out of the question. It must be recognized that reduced service can be expected from a new coating applied after substandard surface preparation, and strenuous efforts should be made to circumvent this situation. However, where it arises, use of method D cleaning may be the next best alternative. This consists essentially of method B plus the use of a metal conditioning solution prior to application of the paint. The metal conditioner phosphoric acid solution tends to consume residual rust and deposit a thin iron phosphate salt on the metal surface; the latter temporarily inhibits rusting of unpainted metal (until painting starts) and may improve bond of, and inhibit rusting under, the applied paint.

The preparatory method B cleaning previously discussed must be the best possible. In this connection, and especially where limited areas require touchup painting of an otherwise generally good coating, we suggest that procurement of the Aurand metal cleaning tool may be in order. While results from this tool do not equal those from blasting, it does a thorough job and has been recommended by Bureau offices using it. Operation and Maintenance Equipment and Procedures Bulletin No. 36 gives more information on this cleaning tool.

An auto body sander has also proven satisfactory for small areas. The coarsest available grit should be selected to provide a roughened surface and minimize a filling of the sanding disks. Sanding has the advantage of producing a bright metal surface, except in pits, and hence a better bonding surface.

Concrete surfaces correctly prepared for painting will usually have sound concrete exposed and be free of loose materials such as old paint, dirt, deteriorated concrete, efflorescence, and any glaze. This may be accomplished in a variety of ways described in the *Paint Manual*. Again, high-pressure water blasting may be appropriate, as discussed later. In maintenance work, the need for exposing sound concrete may be most important; also, curing compound must often be removed prior to any painting; remnants of old joint filler (the bulk of which has otherwise been removed) must be cleaned from grooves. Sandblasting usually best accomplishes these operations. Water-acid solution pretreatments neutralize alkalinity and etch a glazed surface. The degree of dryness the concrete surface must have when coated will vary widely depending on the type of paint selected; this must be controlled by moistening or drying procedures for best coating results.

Wood surfaces on which coatings have deteriorated sometimes present difficult and diverse surface preparation problems. Exterior paint which has peeled or lifted is an example. In such cases, seeking and remedying the cause of any abnormal coating behavior should be the first and most important step. For instance, the source of moisture in wood generally must be eliminated before any oil base paint can be expected to perform properly. Thereafter, areas where the paint has lifted, or is beginning to do so, should be stripped by torching and scraping or other equally effective means if permanency is expected of a newly applied paint.

High-pressure water blasting is a relatively new surface preparation method in that equipment for it is now becoming fairly widely available. A trailer-mounted rig about the size of a 250-cubic-foot-per-minute compressor can produce pressures up to 10,000 pounds per square inch which, for example, are capable of cutting into concrete or cutting through rubber boats and into the bones of the foot. Water blasting (at lower pressures) has been used for cleaning lifts of dam blocks preparatory to the next pour (6,000 lb/in^2), cleaning the upstream face of a small dam prior to application of a coating (2,000 lb/in^2), and cleaning aluminum paint before applying maintenance topcoats (500 lb/in^2). It has been used by others for extensive cleaning of masonry buildings and for a wide diversity of other purposes. A major advantage of this method is that water is a highly available and disposable, ecologically acceptable "abrasive" under most circumstances.

It now appears that water blasting will prove to be a fast and powerful tool for several uses because of the flexibility inherent in the wide range of available pressures and fan or jet nozzles. It should be especially suitable to maintenance work requiring the equivalent of method B cleaning (but not method C, sandblasting), particularly for the following purposes:

1. Cleaning existing paint for repainting. In combination with chemical precleaners as needed, water blasting should remove dirt, scums, and scales and probably poorly bonded old paint.

2. Cleaning concrete or masonry for improved appearance or preparatory to painting. Water blasting should remove most kinds of foreign matter, including poorly bonded old coatings and deteriorated concrete.

It is suggested that renting a water blasting outfit be considered when this method seems appropriate. This office will be glad to provide the latest available data and discuss any particular potential water blasting situations.

In summarizing, the surface preparation operation is considered at least as important a part of painting as the phases later discussed, but one which is often slighted because of the difficulties involved or through lack of understanding of its function. Standards for such cleaning as required for Bureau paints may be found in the C-126 guide paragraph.

Mixing or Other Preparation of the Paint

One would not deliberately tamper with the formulation of a paint and then expect the service normally given by that material; such tampering will usually decrease its serviceability. Obviously, then, strenuous efforts are warranted to avoid accidental and haphazard alteration of the paint ingredients by careless mixing or proportioning. The intent should be to preserve the balance of vehicle, pigment, and thinners as originally supplied by the paint manufacturer. Simple mixing techniques will suffice, involving cutting any settled materials from the bottoms of containers, stirring them until a smooth consistency is achieved, and boxing the paint to insure that all constituents are uniformly distributed. Mechanical mixers facilitate this tiring work and usually do a better job. A stirrer in an electric drill works well. Some paints which settle rapidly require frequent reagitation.

Thinning of some paints is permitted, usually to a maximum of about 1 pint per gallon. However, the practice is to be discouraged. In general, a strong tendency exists toward excessive and improper thinning because it eases the application burden, even if at a cost of decreased serviceability. Much more desirable alternatives to thinning include warming the paint in cold weather and providing the specified equipment (such as the correct spray gun tip and air cap sizes). At other times there is no acceptable alternative to the arm-breaking technique of brushing a somewhat viscous paint.

Some recently developed paints, now coming into widespread use, have added a new complication to paint preparation. These paints set largely or completely by chemical reaction, which reaction begins when a "catalyst" or "accelerator" is added to the paint. Thorough distribution of this additive into the paint is imperative and halfway measures will not do. Also, thinning beyond that recommended by the manufacturer may ruin the paint and cannot be tolerated. In passing, it should be noted that such paints become useless after a stated "pot life" has been exceeded and then must be wasted.

Application of the Paint

Some critical characteristics of a newly applied paint film include its adhesion to the surface, thickness, continuity, and degree of dryness or cure. Application techniques are directed toward meeting the standards of these properties.

Physical adhesion is enhanced if the surface is roughened by some means such as sandblasting. Chemical adhesion is largely a property of the paint; it may sometimes be improved by use of metal pretreatments (conditioners); it will be destroyed by contaminants such as water, oil, or dust on a surface. Chemical adhesion of some paints may also be reduced if low metal and paint temperatures prevail during application.

Brushing the first coat is required for most paints since working the paint tends to insure intimate contact of paint and metal and it also stirs minute amounts of dust into the paint and away from the surface, thus minimizing the threat to proper adhesion. Brushing must be skillfully done if all the surface is to be covered with a reasonable degree of uniformity. A tendency to brush most paints too thinly may be recognized as poor economy since a given dry film thickness is the objective; a thin first coat may cause a later additional, and otherwise unnecessary, coat to achieve this thickness. Paints vary in their brushability. For instance, while house paint dries quite slowly and is easy to apply, VR-3 vinyl resin paint dries so rapidly that a brushful most be spread uniformly at the desired thickness and the painted surface then left undisturbed. Thus, techniques must be adapted to the materials.

Roller coating, where applicable, accomplishes much the same desirable effects as brushing and, in addition, usually provides greater uniformity and speed.

Spray painting, usually specified or permitted for second or later coats, offers the considerable advantage of relatively uniform application of thicker and more continuous films at substantially higher rates. On the other hand, spraying properly requires specialized skill and the correct equipment in good condition. Spray equipment is available for spraying nearly all Bureau-specified paints, from the water-thin silicone solutions up through coal-tar epoxy; most of these can be sprayed using the common pressure pot gear. However, adherence to the recommended gun part sizes for each type must be observed and the equipment cleaned after use. A highly skilled spray painter can be recognized by this technique: gun held 8 to 10 inches from and normal to the surfaces, lapping uniform (such that measurement shows little variation in thickness), systematic pattern for coating an area, and the absence of several bad habits typical of mediocre sprayers. Skilled sprayers will also understand correct adjustment of the equipment to give a properly atomized spray fan, uniformly distributed and without excessive overspray, and they will be careful to see that the paint is deposited as a wet coat, never partially dried before it reaches the surface.

The overall competency of a spray painter influences the quality of the applied film, especially in coating irregular shapes where paint thickness cannot readily be measured. Continuity of most Bureau coatings is insured by specifying that three or more coats be applied but, even then, careless workmanship may cause skips or thin areas. Coal-tar epoxy, red lead, aluminum topcoats and all vinyl materials can be sprayed with conventional pressure pot equipment. Spray techniques and equipment assume special importance with some of the new catalytically set paints which require cross-spraying of two passes with a time delay between; manufacturers' recommendations must be followed rigorously with these materials.

There are other application methods, uses of which are quite limited in maintenance painting, and these will not be discussed here.

Coal-tar enamel ordinarily requires little maintenance, and that usually during the first years of service. As noted in Designers' Operating Criteria, maintenance of enamel can be performed temporarily with coal-tar epoxy, but a permanent repair should be considered for about the 10th year with additional enamel if a sizable area is defective. At this time, services of a qualified enameling contractor can be secured for any major enameling which usually requires specialist services. If the area involved is small enough, the work could be done satisfactorily by project forces, but this is rarely attempted. Enamel application is described at length in the *Paint Manual*, and because of its complexity, must be eliminated from this discussion. However, the recent adoption of a *synthetic resin primer* for enamel is a *substantial improvement* in this coating and deserves special mention. This primer promises to decrease both application and maintenance problems because the initial application is more likely to be well bonded and to resist deteriorating forces prior to operation of the structure. The extent of repair to enamel coatings should be very small, but if it becomes necessary, the new primer will greatly facilitate effective repairs under adverse conditions.

Coal-tar epoxy paint is now coming into widespread use. It was specified to a limited extent for Bureau metalwork for a while, but is now employed generally in most exposures where CA-50 formerly was required. It is, of course, expected to provide much longer service than CA-50. Coal-tar epoxy paint is a black, two-component material having application characteristics similar to CA-50, and is applied according to the manufacturer's instructions with certain exceptions shown in the C-126 guide paragraph. Some features of the application are especially critical. First, *mixing* must effect thorough intermingling of the two components which react chemically to set and cure the paint. Next, while application may be by brush, spray, or roller, the *roller* is the most practical method for all coats on a moderate size job. Areas inaccessible to the roller may be brushed, and spraying is entirely feasible provided the manufacturer's instructions relating to spray gun part sizes are followed closely. Also, correct time between coats has been found vital to proper application; delamination (disbonding between coats) may occur if the undercoat cures too hard before recoating takes place. The topcoat may be applied as soon as the undercoat becomes tack free, but the time before topcoating should not exceed 50 percent longer than the firm-to-touch time. These times may vary quite widely depending on ambient air and metal temperatures. Finally, an adequate period for curing should be programed, as discussed below.

Coal-tar epoxy paint may be particularly advantageous in maintenance painting where short outages and humid conditions prevail. In common with other epoxy paints, it is said to tolerate surface contaminants, including moisture, better than other immersion paints. Moreover, heat can often be applied to small areas, thus decreasing the time between coats to 2 to 4 hours and effecting a quick, partial cure of the finished system.

Curing

Most paint films fail to develop their full protective capabilities unless adequately dried or cured. It is realized that schedules for some operations can severely restrict time available for painting; however, the effort expended in painting will be largely or completely wasted unless sufficient time is provided. Planning well in advance should aid in scheduling adequate time for curing or in permitting acquisition of special equipment such that the cure can be forced. Introducing heat or brisk air circulation can expedite curing materially. Where items too large to heat are exposed to winter temperatures, the work can sometimes be scheduled for a seasonal outage of long duration for curing. Time-temperature tables govern the curing of the chemically set paints and must be followed carefully; however, these paints will usually be impractical if the temperature during curing cannot be held above the stated minimum (usually about 50 °F).

Inspection

Inspection of painting may not always be formally conducted in maintenance work; its intended function should, nevertheless, be served in some manner. The tests to be made are few and relatively simple in nature, but they will aid substantially in achieving the end results.

Recalling that surface preparation is an integral part of painting, the condition of the cleaned surface can be examined to determine if all foreign matter has been removed by blasting or loose material has been wire brushed adequately and that residual products such as blast dust have been taken care of. Paint coverage may be determined while the paint is wet by using a small gage. Another gage will show thickness of a dried film on a ferrous surface. Supervisors should also note and correct poor painting practices that lower the effectiveness of the work.

Summary

Limitations have restricted the above discussion to stressing the most important and basic principles of painting, an attempt being made to dwell on the complications introduced by maintenance circumstances. We emphasize the importance, first, of searching out and meeting the standards for surface preparation and application methods for the paint selected and, second, of bending every effort to create adequate temperature, humidity, and ventilation conditions for these operations to proceed.

Guide paragraph C-126, previously referred to, shows current coating selections for various exposures and gives specific surface preparation and application requirements for Bureau paints. Procedures for each operation, as well as comments on the characteristics of Bureau-applied paints, are detailed in the Paint Manual which is recommended reading when painting is planned. It is recognized that most maintenance painting involves special individual problems which could not be covered in this discussion. Assistance with such problems is available through usual Bureau channels.

Water Systems Operation and Maintenance Workshop - 1990

VEGETATION MANAGEMENT AND PEST CONTROL, PARTS 1 AND 2

Session 12-1 - Carl Tennis
Session 12-2 - Dave Sisneros

Introduction

Management of weed and other pest problems on water systems in the Western United States has moved through several phases. The early period beginning in the late 1800's was characterized by a lack of effective and economical control methods; delivery of irrigation water in the face of severe problems was accomplished only through great effort with crude manual and mechanical practices. As machine power was applied generally in agriculture during the 1920's and 1930's, it was also adapted to irrigation systems. This phase probably reached its peak in 1950.

Starting at that time, a shift away from labor and energy intensive practices occurred. Planting desirable vegetation on ditchbanks was recognized as an effective means to provide competition to weedy plants. Also, modern herbicides developed following World War II were adapted to water system use to provide selective or total control of objectionable land and aquatic weeds. This trend continued to the near exclusion of mechanical practices by the 1960's.

However, recent public concern for protection of the environment has resulted in the enactment of Federal and state laws that increased regulation of pesticides including herbicides. By intent and through delays in implementation of these regulations, there has been a severe restriction in the number and latitude of chemical control techniques available to project operators, particularly for uses in and around water. An understanding and working knowledge of these laws are essential to ensure compliance and have become an integral part of vegetation and pest control in the management process.

When the legislation is fully translated into practical regulations, many new materials will be registered for water systems, and pesticide use will occupy an important role. Management of the biological influences on water systems will involve use of these new tools and some of the old herbicides. This will be part of an increasing reliance on an integrated approach using all kinds of technology - including mechanical methods, biological control agents, and manipulation of the environment of water facilities.

Vegetation Management

As used now, the term vegetation management includes all the planning, preparations, and actual practices which an operating entity may use to minimize potential adverse effects of vegetative growth on and in its channels and other facilities. It is also appropriate to define it as including those elements which maximize the efficiency, economy of operation, and esthetic qualities of these works. The many aspects of vegetation management and pest control on irrigation and multiple-use water systems represent one of the most expensive items in the operation and maintenance budgets for these facilities, averaging 20 to 25 percent of the total costs.

A well-planned vegetation management program includes weed prevention. It will be based on the premise that practically all parts of a system, excluding rocky and other inhospitable areas such as lined reaches and paved roads, will be subject to habitation by plants. The logical action is to guide the result so that permanent plant population will exercise the least adverse effect on the facility. This can be accomplished by planting and encouraging species that are adapted to the local environment and have qualities compatible with operation of the system.

Weeds

Weeds are defined as plants out of place or plants that exercise a negative effect on man or his environment. Even the best stands of competitive vegetation will suffer some invasion by aggressive weeds. Use of herbicides, mowing at a particularly effective stage, or even fertilization to tip the balance in favor of the desirable species are examples of weed control. Control of a pest becomes necessary when its actual or potential level of infestation passes a threshold beyond which unacceptable economic or other damage will result.

A third part of management may be called recovery. This phase is required when a combination of prevention and control fails to the extent that the pest exercises a detrimental effect on operation of the facility. Examples of weed recovery costs are: personnel and equipment time spent removing and disposing of debris, repair of ditches and structures washed out due to plugging, and loss of delivery capacity from flow impedance by aquatic weeds. Costs required to recover from the effects of weed growth are often overlooked and hidden in other operation and maintenance costs.

Prevention of weed costs is the most economical of the three phases - recovery from the effects of runaway infestations is the most expensive. Some fundamentals and principles of better vegetation management and pest control will be reviewed in this session. Also, some new ideas and approaches to controlling unwanted insects and animals will be covered in the limited time available.

Prevention

Weed prevention methods include procedures aimed at limiting the spread and establishment of weeds. It would be poor economy to base a program entirely on continual control and eradication without including plans for weed prevention.

Knowing how weeds may find their way to ditchbanks and using reasonable care wherever possible to prevent infestations from these sources will save much labor and expense.

Nature plays an important role in the spread of land weeds by wind, water, and animals. Many weed seeds are equipped with cottony appendages or parachutes, such as canada thistle, which allow them to travel great distances by wind and eventually lodge on ditchbanks or nearby farmland. Tests have shown that some weed seeds are carried by water. Many ditchbank infestations have started from seed originating in watersheds, around margins of lakes and reservoirs, or along rivers and streams. Both wild and domestic animals carry weed seeds on muddy feet and in wool and hair.Birds move seeds of aquatic plants by way of their digestive tracts. People also spread weeds to ditchbanks through planting grass or crop seeds which contain weed seeds, and by obtaining repair or fill dirt from areas adjacent to ditchbanks, which are infested with noxious weeds.

Aquatic weeds are spread to irrigation ditches mainly by the water from sources above the project. As well as from seeds, some water weeds may start new infestations from vegetative parts, such as pieces of stems and roots, and winter buds and tubers which break loose and finally lodge in a suitable growing site. Both wind and water play an important part in the spread of algae spores. The most effective means of preventing both land and water weed infestations on ditches is to eliminate the source. There are many instances where the source can be controlled or eliminated, and even though the initial cost is comparatively high, much labor and expense can be saved in the long run.

Years of experience in operating and maintaining irrigation projects indicate that certain provisions in designing and constructing ditches should be included to facilitate weed control. The ditchbanks should be constructed with flat slopes and smoothed to furnish a good seedbed for weed-competing perennial grasses. Ditchbank roads are essential for proper inspection of the system and for easy travel of weed sprayers and mowers when necessary. The planting of desirable sod-forming perennial grasses at the waterline and adapted species on the dry zone is the most successful means of preventing land weeds on ditchbanks and costly wind and water erosion. The full realization of the magnitude of the ditch weed problem, and the operation and maintenance costs necessary for controlling weeds once they become established, should create an interest in devising every possible method of preventing future infestations. The means by which weeds obtain their foothold on ditches offer clues to weed prevention; therefore, every operator of an irrigation system should determine these means for his own area.

Control

Methods of controlling unwanted vegetation are classified as herbicidal, mechanical, biological, and environmental manipulation. The use of chemical herbicides and mechanical procedures, such as mowing, burning, and cultivating, are the most common. Cultivation has no application to rights-of-way. Biological control using animals, fish, insects, and disease pathogens is in its infancy and has yet to be developed to an applied level. Manipulation of environmental factors that regulate growth of weeds is being explored. Some of these newer ideas will be touched on in these sessions. Since most present-day weed control programs utilize herbicides, discussions will be directed toward improvement of vegetation management through better use of herbicidal techniques. A review of some of the fundamentals important to effective use of herbicides is presented here to acquaint and refresh session participants with these principles.

Land Weed Control

Classification of Plants

It is important to be able to distinguish between grass and broadleaf plants because they differ in reaction to herbicides and other control methods. For weed control purposes, plants are divided into two main categories: grass and broadleaf. Grass plants have one seed leaf. They generally have narrow, upright, parallel-veined leaves and fibrous root systems. Broadleaf plants have two seed leaves. They generally have broad, net-veined leaves and a taproot or coarse, branched root system.

Weeds are also classified into groups according to their life cycle. Knowing the growth habit of weeds is important to the choice and effectiveness of control methods. The three groups are:

Annuals complete their life cycle from seed in less than 1 year. As a general rule, annuals are more susceptible to kill with herbicides and mechanical methods than biennials and perennials. Winter annuals germinate in the fall, lie dormant over the winter, and complete their life cycle early the next summer. Summer annuals germinate in the spring and complete their life cycle by fall. Among the troublesome annuals on ditches are Russian thistle, the mustards, lambsquarters, kochia, and sandbur grass.

Biennials complete their life cycle in more than 1 year but in less than 2 years. They produce taproots and basal leaves the first year, storing an abundance of reserve food. The second year they make rapid growth and produce flowers and seed. Some of the most troublesome are mullein, bull thistle, and burdock. There are no grasses in this group.

Perennials live for more than 2 years; only the aboveground parts die each fall. They are the most resistant to kill with herbicides or mechanical methods and spread by seeds, roots, and aboveground stems. Simple perennials, such as the dandelion, spread only by seed and have large, fleshy roots. Creeping perennials spread by seed and extension of underground root systems amply supplied with buds from which new plants can be initiated. Some of the most troublesome perennials on irrigation systems are canada thistle, Russian knapweed, reed canary grass, johnson grass, and bermuda grass.

Applying Weed Classification to Control

Grass plants are generally more difficult to control and eradicate than broadleaf plants due to their structure and growth habit. The growing points (sites at which the plant elongates) in grass plants are in the crown, whereas in broadleafs, they are in the terminals of the branches and the points where the leaves are attached to the stem. Grasses recover from mowing, while most simple-rooted broadleafs are killed. Herbicide applications to grasses often fail when the material merely burns off the foilage and is not translocated to the crown and roots. Spraying of perennial weeds with creeping roots must be done so as to utilize the aboveground parts of the plants to move the herbicide to the roots and their rhizomes and dormant buds.

Weeds are most vulnerable when in the seedling stage of growth and are actively growing. Of the three groups of plants, annuals are most nearly like seedlings and are very susceptible to kill with herbicides and mechanical methods. Biennials and perennials are most easily killed if they can be attacked when their development is similar to that of an annual or during their first year of growth. Food reserves in biennials are low then, because the plants have not had time to fully develop their taproots and food stores. Perennials have not developed their extensive root systems in which food is stored and from which the stand can be regenerated by release of dormant buds. After they are well established, perennials are vulnerable only at certain times, such as when nutrients are at low levels following rapid growth, the parts above ground are new and tender, and the physical processes (cell division and movement of food and water in the plant) are in full swing.

Factors Which Influence Effectiveness of Herbicides

Knowledge of plant, environmental, and herbicide factors which determine the success of herbicide application is essential for effective and economical maintenance spraying. With a large acreage to spray, a district cannot cover all rights-of-way at the optimum time; however, consideration of changes in plant and environmental conditions, as the season progresses, can provide a basis for maintaining high returns on the material, labor, and equipment expenditure. If control cannot be applied at the optimum time, then the herbicide, rate, or method may need to be changed.

Plant Factors

The physical characteristics of a plant influence retention, penetration, and intake of an herbicide. Leaf shape, leaf position, type of leaf surface, and the density of leaves all play major roles in the problem of getting sufficient herbicide onto a plant to kill it. Also, the physical condition of a plant is important. Collectively, these factors act as a group of "obstacles" to successful herbicide application. Herbicides should be applied in a manner to bypass these obstacles in order to achieve weed control.

Leaf Characteristics

Leaf characteristics, contour, and habit affect spray placement and retention. Horizontal leaves retain more spray than vertical leaves, and rough, hairy surfaces retain more than smooth surfaces. Air is trapped between the spray and leaf surface on some leaves with pubescent (hairy) surfaces, such as mullein, or with ridges, such as on grass leaves, figure 1. Folding of leaves under low moisture conditions decreases coverage.

Character of the Cuticle

The thicker the surface cell layers and waxy covering on the leaf, the less penetration of the spray takes place. Cuticle differences are often indicated by the way the leaf surfaces are wetted by the spray liquid.

Distribution and Size of the Stomata (Pores)

Deposits of spray on stomata-bearing surfaces are desirable; tests indicate 15 percent of spray penetration is through these openings. Most leaves have stomata on both upper and lower surfaces, but more on the lower. Complete leaf coverage is desirable.

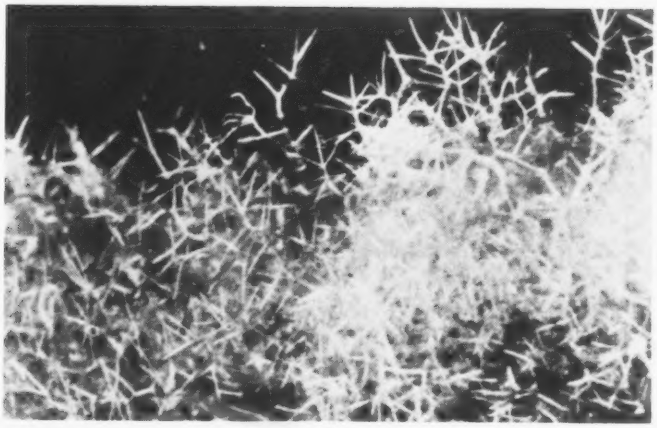

Figure 1. - Leaf from mullein plant. Note pubescent, hairy covering which suspends spray droplets away from leaf.

Stage of Growth

Young leaves are easier to penetrate than old leaves. There are also seasonal variations in the movement of fluids in plants. Movement is most complete in young, active plants. This movement slows as the plant matures. Thorough translocation of herbicides in plants is closely related to nutrient and water transport. Biennial and winter annual weeds are hard to kill after they go into the "bolt" stage of rapid growth and food movement is largely from reserves. Most perennial weeds translocate herbicides best after expenditure of food reserves in rapid spring growth and during replenishment of those reserves.

Environmental Factors

Light

Light is one of the basic requirements for plant metabolism. A plant made active by adequate light conditions will be subject to more herbicide penetration than an inactive plant. The stomata will be open for the exchange of gases. Translocation of a herbicide will be more efficient because the plant is manufacturing and translocating food. Long days of 16 hours are more favorable for 2,4-D toxicity than short days due to the extended plant activity.

Adequate Supply of Soil Moisture

An adequate supply of soil moisture producing a favorable water balance in the plant improves absorption and movement of herbicides. It softens the cuticle, supports food manufacturing and translocation, and speeds up enzyme reactions and plant growth.

High Humidity

High humidity reduces transpiration water stress in the plant and delays drying of the spray deposit. It also favors opening of stomata and permeability of the cuticle.

Rain

Rain before a spray application will wash off and refresh the plants so that kill is enhanced. Heavy rain soon after spraying can wash off the herbicide deposit. Light rain may be a benefit by extending or renewing the aqueous contact of the spray solution with the plant. Steady, slow application of water, as from a moderate rain, is necessary to leach in and activate soil-applied herbicides.

Dust

Dust on plant surfaces may soak up and dry herbicide solutions quickly so that penetration and absorption are prevented. Some herbicides are tightly bound to soil particles upon contact.

Hot Winds

Hot winds will cause plant pores to close, leaf surfaces to thicken, and wax layers of the cuticle to harden, thus reducing effectiveness.

Variation in Annual Weather Conditions

If an organization has a large acreage to spray each season, there is a temptation to start the operation by a fixed date instead of basing it on the growth stage of the weeds. Particularly with annual weeds, an examination should be made to ensure that there is adequate and uniform germination and emergence of the target species to get the bulk of the eventual stand. A premature spraying will have no effect on seedlings that emerge afterward.

Herbicidal Factors

Spray Solution

The spray solution is the starting point for penetration, absorption, and eventual translocation of the herbicide in the plant. Movement of the herbicide through the plant's cuticle is accomplished by the transfer of the herbicide from the solution of the droplet to the solution within the plant. The longer the spray remains liquid on the leaf, the better the penetration and transfer. The pH (acidity-alkalinity) and buffer composition of the solution can determine the effectiveness of the herbicide. Many weak acids enter cells more readily than alkaline solutions. Penetration of 2,4-D is better at pH4 than at pH7. The chemical structure of the herbicide has a direct effect - ester derivatives of 2,4-D are more effective than salts because their volatility results in better penetration. However, that characteristic is often the cause of damage to nontarget plants.

Coverage

Coverage of all the foliage of individual plants and all plants in a stand ensures adequate exposure of the weeds to the herbicide. Coverage of all plants is especially important with perennials as all the aboveground shoots may be attached to the common colonial root system. Thorough coverage will ensure movement of herbicide to all parts of that system and its dormant buds. Except where high gallonage is needed on heavy foliage, booms sprayers achieve better and more uniform coverage than handguns. A boom sprayer driven at a known speed while applying a known amount of spray per minute will apply a precise rate of herbicide.

Use of handguns is appropriate where the weeds occur in scattered patches or heavy concentrations, and high rates of solution and herbicide are required to obtain kill.

Additives

Various additives are included in a pesticide to facilitate or enhance the effectiveness of the active ingredient. Any given pesticide contains one or more active ingredients and an inert diluent or carrier. Several additives may be mixed in at the time of manufacture; others may be added or increased at the time of use. With 2,4-D, it is important to add more surfactant. Since most 2,4-D formulations are intended for agricultural crop use, large enough quantities of surfactants for noncrop weed control are not included. This is required to avoid reducing or eliminating the selectivity of the product in crop use.

Molecules are the submicroscopic "bricks" of which all things are made. Each molecule exerts a "pull" similar to a magnet. Water has surface tension because of its molecular structure which causes each water molecule to have a strong attraction for other water molecules. Molecules on the surface are pulled inward because there are no water molecules on the other side to exert force. This inward force causes water to stay in the smallest possible area, which is a sphere or drop. In other words, surface tension causes water to form a "skin" and makes it form drops. Surfactants are added to overcome surface tension and to make sprays more effective in other ways, figures 2 and 3.

"Surfactant" is a coined word which combines the words "surface active agent." It is easiest to visualize the action of a surfactant as a chemical that reduces surface tension and allows water to spread out. Surfactants are also called adjuvants, spreader-stickers, and wetting agents. However, the last two terms are inadequate for they describe only some of the effects which a good surfactant will provide. A full list of surfactant actions would include:

1. Improve coverage by reducing surface tension of the spray droplet
2. Remove air films between the spray and leaf surface
3. Improve cell wall penetration
4. Act as a cosolvent of waxy layers
5. Increase entry through stomata (pores)
6. Act as a humidifier to maintain longer wetting

In a university experiment, plants absorbed three times as much 2,4-D in 72 hours with a surfactant in the solution than without it.

Mixture	Percent Absorbed
2,4-D alone	10.1
2,4-D plus 0.1 percent surfactant	36.5

Wetting agents may lower the selectivity of a weed killer while increasing its effectiveness. The composition and concentration of the surfactant determine its effectiveness. A certain material at a given rate may be

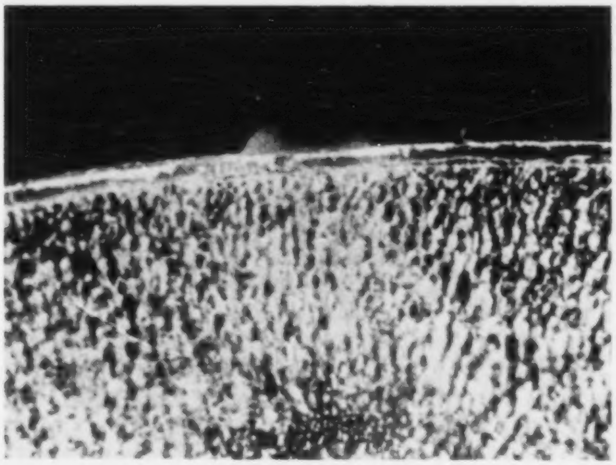

Figure 2. - Cut edge of leaf. Water is a poor wetting agent. Herbicide in water alone, without surfactant, is held in droplet on the waxy surface of leaf. The result is poor absorption of the herbicide and, consequently, results in poor weed control.

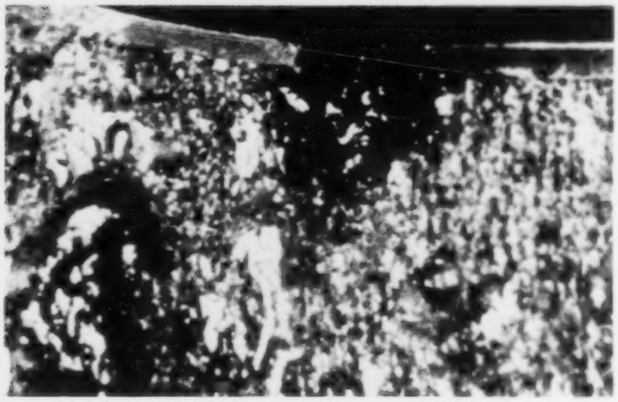

Figure 3. - Cut edge of leaf. The waxy cover on the leaf surface can be penetrated. High rate of surfactant added to drop of water and herbicide cuts the wax.

ideal for one use and ineffective for another. One herbicide comes with a special surfactant packed separately for addition at mixing of the herbicide. Sometimes the use of too much surfactant increases runoff of the herbicide, resulting in a smaller spray deposit, figure 4. Use of the proper amount may permit reduction of the amount of herbicide in the mix through the increase in effectiveness.

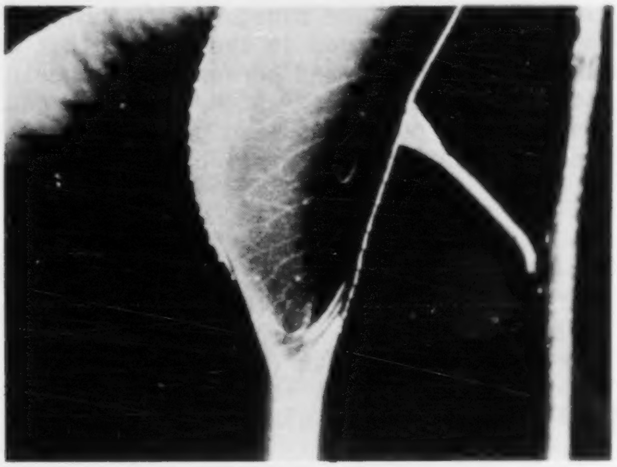

Figure 4. - Leaf surface. Note runoff of herbicide - do not use too much surfactant.

Herbicides

Herbicides are classified in several ways in order to define them and to describe their method of use.

1. Mode of Application

Foliage applied means herbicides applied to the aboveground parts of plants. They include oil solutions and emulsions used on the bark of brush and trees in a dormant application. Examples are glyphosate and 2,4-D.

Soil applied includes herbicides applied to the soil in order to make it physically hostile to plant growth or to be absorbed by the plants' roots from the soil solution. Some foliage-applied herbicides may act through the roots when washed into the soil. Examples are Diuron and Sulfonylureas.

2. Mode of Action

Contact herbicides are applied to weeds after they have emerged from the soil. They kill only the foliar parts of the plant and do not directly affect plant roots. With seedlings, annuals, and young biennials or perennials, this may be enough to kill the plant. Contact herbicides commonly used are weed oils, diquat, and xylene.

Systemic herbicides may be foliage or soil applied. They are usually applied in low concentrations, absorbed by the plants, and moved through their systems. They upset the normal functions and balances within the plants, thus leading to injury and death. There are many kinds of systemic or translocating herbicides, with 2,4-D and glyphosate being the most commonly used.

Growth inhibitors, although they are not true herbicides, should be mentioned in connection with vegetation management. Applied as foliage sprays, they cause physiological changes in the plants that prevent normal growth, such as reducing plant height and preventing flowering and seed production. They seldom kill the plants. Such compounds could be very useful in reducing the vigor of perennial weed grasses on ditchbanks and still allow the grasses to stabilize the canal banks.

3. Selectivity

Selective herbicides will control some plants with little or no effect on others. The removal of broadleaf plants with 2,4-D from a grass stand with no damage to the latter is a good example. Nonselective herbicides kill all plants; the herbicide glyphosate (Roundup) appears to be a systemic nonselective herbicide. Diquat is used as a contact nonselective herbicide. Selective herbicides may be classified into two groups:

> Broadleaf herbicides are effective on broadleaf or dicotyledonous plants (meaning two seed leaves). They will not control grassy weeds. Typical examples are 2,4-D, dicamba (Banvel), and picloram (Tordon), to name a few.

Narrowleaf herbicides control grass plants or those that have only one seed leaf (a monocotyledon). There are fewer herbicides that have activity on grass plants than on broadleaf plants. Dalapon (Dowpon) is an example of a grass herbicide that has been used widely on irrigation systems.

4. Chemical Nature of Herbicides

The kind of active ingredient an herbicide contains determines its activity on plants. A given basic material may be structured in a variety of ways. For example, the acid of 2,4-D has been used directly; however, more effective forms were developed. Some of the common derivatives are: sodium salt, ammonium salt, amine salt, and a variety of esters of 2,4-D. Even slight changes in the chemical structure may give striking differences in the plant-killing qualities of the resulting material.

5. Formulation of Herbicides

Herbicides may take a given physical form as a consequence of their chemical makeup or they may be prepared in a specific form to facilitate application.

Soluble Forms

The chemical can be dissolved in water or oil. The resulting solution does not separate into solute and chemical if allowed to stand, and after initial mixing, agitation is not required. Most herbicides fall into this category.

Emulsion

This type of preparation permits mixing oil-like chemicals with water. The emulsifier may be added to the herbicide at manufacture or just before use as with xylene. The better the grade of emulsion, the less agitation is required to mix it with water. A good emulsion is cream colored and does not separate into its components for a considerable time.

Wettable Powder

This type contains a pesticide that has been mixed with or sprayed on a carrier powder. The entire mixture has been treated with a wetting agent so that it will go into suspension in water readily. Spraying of wettable powders requires constant agitation of the water carrier. The solubility of the herbicide determines how quickly it is freed from the carrier and the depth to which it is leached into the soil with a given amount of precipitation, thereby determining the depth of the soil treated. Many soil-applied herbicides are prepared as wettable powders.

Granules and Pellets

The herbicide is incorporated with clay or another dry diluent to facilitate spreading a small quantity of active ingredient over a large area by hand or machine application. The release and action of the herbicide are the same as with wettable powders. Granules and pellet forms are widely used for soil-applied materials.

Mechanics of Spray Production

Many kinds of nozzles have been developed for breaking up liquids into spray droplets for uniform distribution over the target plants. All of these devices apply energy in the form of pressure on the liquid and use the resulting motion to disperse it. Factors governing the spray droplet size and pattern are the velocity resulting from the pressure, surface tension of the liquid which is a function of its viscosity (resistance to flow), and the design of the device through which the liquid is emitted.

Sprays are produced by two basic processes: aerodynamic breakup and instant atomization. In the first process, the spray issues from the nozzle in a solid stream or a flat or hollow-coned sheet. The form of the fluid maintains its integrity at first, but air acting against it strips the stream or sheet apart into primary droplets. As these travel farther, air resistance overcomes surface tension and pulls them apart into smaller droplets. The eventual size of the particles is determined by the initial velocity. Aerodynamic breakup is illustrated well by the droplets and pattern produced by a handgun set at solid stream adjustment. Also, at low pressures, fan and hollow-cone patterned nozzles produce their sprays in this same way.

At high pressures, nozzles produce the spray through instant atomization. The pressure and the nozzle orifice operate to shear the fluid immediately into small droplets. This mode of spray production is the least suited to weed spraying because of the higher proportion of fine droplets produced. The drift hazard of the herbicide to nontarget plants is directly proportional to the amount of fines contained in the spray pattern. With equal air movement and height of release, a small droplet will travel farther than a large one.

Three types of boom nozzles are used commonly for right-of-way weed spraying. All of them produce spray droplets by aerodynamic breakup, although they differ in design and the way that the initial fluid sheets are produced. Based on the mode of operation, the types are:

Forcing liquid through a small, structured opening - e.g., flat-fan, even-fan, and off-center nozzles
Deflecting a stream of liquid off a plate into a fan pattern - e.g., floodjet nozzle
Swirling the liquid out of an orifice into a hollow-cone pattern through circular motion produced in a cylindrical chamber - e.g., whirl chamber nozzle (hollow-cone patterns are also produced through use of swirl plates ahead of an orifice)

A summary of the characteristics of these types follows:

Flat Fan

Comparatively high pressure required; design pressure, 40 pounds per square inch (best performance obtained at 30 pounds per square inch or above; not recommended for less than 20 pounds per square inch)
Small orifice clogs easily
Drift potential increases at pressures above 30 pounds per square inch
Pattern good at 30-inch height
Orifice subject to rapid wear and mechanical damage
Spacing - 20 inches
Cost - low
Installed in vertical position

Floodjet Nozzle

Low pressure - 10 to 15 pounds per square inch
Clogging minimized by large orifice
Low drift potential from coarse spray
Pattern good with overlap, fan shaped
Durability - excellent; orifice protected from damage
May be spaced at 40 inches
Cost - slightly higher than fan nozzle
Installed in horizontal position

Whirl Chamber Nozzle

Low pressure - 8 to 12 pounds
Virtually free of clogging due to large orifice and absence of swirl plate
Low drift potential at proper pressure
Pattern - excellent; hollow cone covers top and all sides of plants
Durability - good; outer orifice subject to mechanical damage
Spacing - 20 inches
Cost - approximately double that of fan nozzle
Nozzle orifice plates may be changed to vary capacity

Clogged Nozzles

Clogged nozzles use more crew time than any other sprayer malfunction; they also result in poor control by leaving strips of untreated weeds. The size of the nozzle orifice directly governs frequency of clogging - select a large one to reduce clogging. A larger proportion of particles which may plug the orifice will pass through with the solution.

Prescribe for Each Situation

The physician selects from a wide array of treatments and medicines for a particular disease. Likewise, the weed expert or the staff assigned to weed control should prescribe for the particular problem to be solved or job to be accomplished. Following is a partial list of factors which should be considered in choosing methods and materials:

1. What will be the consequences of not acting? Will the infestation proceed to a stage requiring recovery costs? Will they exceed the cost of control?

2. What treatment options are available? If a poor return from herbicide application is probable, would another kind of treatment, such as mowing, be a better solution?

3. Is the objective eradication, control, or suppression?

4. Is there a mixture of plants in the area? If so, are some desirable or nonobjectionable?

5. Are the target weeds broadleaf or grass plants?

6. Are they annuals, biennials, or perennials?

7. Do you wish to exercise selectivity and leave certain species while eliminating others?

8. What are the stages of growth of the target species? Are they young and actively growing or old and in slow growth?

9. Are you timing the application to best control the target weeds?

10. Are weather conditions in your favor?

11. Is the proposed herbicide treatment in compliance with Federal and state laws? Has full consideration been given to protecting crops and other desirable vegetation, water quality, and other environmental elements? Read and follow the herbicide label to obtain maximum effectiveness and to protect nontarget organisms. Caution: A herbicide must have label instructions which permit the intended use.

12. What changes are suggested that may prevent the infestation in another year?

Administration of a Successful Weed Control Program

An irrigation superintendent and the staff members that are responsible for weed control should establish the objectives of weed control they hope to accomplish at the start of a program. Once program objectives have been established, an adequate training program should be initiated to enable weed control personnel to conduct the program or qualify for any required licenses. Formal training can consist of sending the weed control staff to workshops and training sessions conducted by Bureau weed control specialists, weed control schools conducted by state university or extension service, and weed district meetings. Self-improvement should be encouraged to collect and study the weed control publications, many of which are available from the Bureau, Department of Agriculture, state colleges and universities, and chemical manufacturers involved in research.

Prior to starting field operations, the weed control staff should determine the kinds (species) of weeds which should be controlled, what methods of control that are to be used (mechanical, chemical, etc.), and equipment needed.

An important factor to be considered prior to starting field spraying is the calibration of weed sprayers or applicators. Many times this small but important operation is overlooked. Accurate sprayer calibration will assure that enough herbicide is being sprayed on weeds to get good control. Yet, it avoids the use of too much material which may be harmful to other desirable vegetation on your ditchbanks, to adjacent farm crops if drift occurs, or cause pollution of irrigation water. Applying excessive rates is expensive as well as creating undesirable effects on the environment. A thorough knowledge of what the transported water is being used for and where drainage waters will go is necessary. (Usage of water such as irrigation of farm crops, pasture, domestic water supply, or watering livestock and the approximate amount of water that will flow through the canal, lateral, or drain at the time treatment is to be made must be considered.)

All weed control operations must be timely to be effective. Usually the period of time that is considered ideal for treatment of the different kinds of weeds is short. It is, therefore, necessary to have all phases of the program well organized, have all chemicals on hand, and equipment in a good state of repair so that valuable time is not lost when the time is right for treatment of a particular weed or weeds.

Following treatment, particularly with chemicals or burning, it is highly desirable to check after a few days to observe results and determine effectiveness. Many times this small but important phase of the program is forgotten or neglected. Careful observations soon after treatment can many times indicate that some modification in the method of treatment is needed or that the treatment was ineffective and should be not be used again.

Keeping complete records of the weed treatment has also proven invaluable in evaluating results and in determining costs. These records can also be very useful in the event of a claim of damages or herbicide label misuse is submitted as a result of weed control operations.

Repeat treatments of noxious weeds on ditchbanks and aquatic weeds in canals and laterals are often required during the same season or the next season. Therefore, it is advisable to use some means of marking these infestations with a painted stake in the field so that weed control crews can quickly find and retreat the regrowth.

During the irrigation season, you may find new land weeds or aquatic weeds which you cannot identify. Without proper identification, it may be difficult to select the proper method of control. Land weeds which you wish to have identified can be taken to your local county extension agent. Since aquatic weeds are not commonly known and may be more difficult to identify, you should furnish a live specimen of the weed to the Bureau weed control specialist or weed extension agent in your area.

Laws and Regulations

The amended FIFRA (Federal Insecticide, Fungicide, and Rodenticide Act) of 1972, its regulations, and subsequent amendments are administered and enforced by the EPA (Environmental Protection Agency) and various cooperating state regulatory agencies. Amended FIFRA and its regulations have superseded many former state statutes and regulations which regulate the registration, formulation, distribution, and use of pesticides.

There have been several amendments to FIFRA since 1972. Several changes were made in the 1978 amendments that impact the use of pesticides by water users. All who apply pesticides or supervise their application should be familiar with the basic requirements of Federal and applicable state pesticide laws and regulations and how to comply with them.

Time does not permit discussion of all applicable sections of FIFRA and its regulations that may directly affect your pest control activities. Let us discuss a few pertinent requirements that may help you in planning and implementing your pest control programs to avoid legal actions and/or complaints from the public.

FIFRA, section 12, is the unlawful acts section. Section 12(a)(2)(G) states that it is unlawful to use any registered pesticide in a manner inconsistent with its labeling. The label is a legal document and if you follow its directions you will be in compliance with FIFRA. The EPA has concluded agreements with most states to enforce FIFRA, including the investigation of misuse.

Use of a pesticide in a manner inconsistent with its labeling is a violation of the law and subjects the misuser to penalties of fines and/or imprisonment. The 1978 amendments relax EPA's previous interpretation of "inconsistent" so now it is permissible to:

1. Apply less chemical per acre, use a more dilute spray solution, or apply less often than suggested on the label (see item 5).

2. Apply a pesticide for control of a pest not listed on the label provided the crop or site is on the label and does not say "for use only against the pests listed." The applicator must abide by all other directions and precautions including maximum rate of application and preharvest interval.

3. An applicator may use any application method not prohibited by the label, except see item 5 below.

4. A pesticide may be mixed with a fertilizer provided such a mixture is not prohibited on the label.

5. The amendments permit use of a pesticide at less dilution (more concentrated spray) for agriculture or forest use after March 31, 1979, unless the EPA administrator issues a regulation or advisory opinion which would require definite amounts of dilution.

The 1978 amendments did not alter the fact that it is still a misuse to apply any pesticide to a site that is not named on the label under directions for use; for instance, applying dicamba to the inside banks of an irrigation ditch, a use site for which it is not registered.

Section 12(a)(1)(b)(A) of FIFRA states that it shall be unlawful for any person in any state to distribute, sell, offer for sale, hold for sale, ship, deliver for shipment, or receive and (having so received) deliver or offer to deliver to any person any pesticide which is not registered under section 2, except as provided by section 6(a)(1).

Section 12(a)(2)(A) of FIFRA states it shall be unlawful for any person to detach, alter, deface, or destroy, in whole or in part, any labeling required under the act.

Here are some "do's and don'ts" that, if followed, may help prevent some problems resulting from herbicide use:

1. Project and irrigation district management should always know where irrigation canals and ditches terminate. This seems elementary, but occasionally management personnel do not know this. Irrigation waters treated with well-known fishkilling chemicals, such as acrolein and xylene, should never be permitted to enter natural streams. Not only is this a violation of the law, but fishkills are very visible to the public. Fishkills are a very emotional issue, and are readily seized upon by environmentalists to fuel their causes and also by the news media. Often the magnitude of such kills is greatly exaggerated, partly due to their rather spectacular visibility. Users must avoid this pitfall or risk having very much needed registrations of these aquatic herbicides further restricted or canceled. Know the capabilities, limitations, and hazards of all herbicides you propose to use and take the necessary precautions to avoid problems.

2. Leaky and other flow-through structures that cross or come in close contact with natural streams should be repaired before applying aquatic herbicides.

3. Treated canals or laterals should be sufficiently patrolled to prevent deliberate or accidental opening of checks.

4. When you purchase bulk tanks or fill bulk tanks with a chemical, assure that the bulk tank is properly labeled with the registered label. If bulked chemicals are transferred to nurse tanks in the field, make sure the nurse tanks are properly labeled with the registered label or that someone in charge of the field application of the pesticide has a loose registered label in his possession.

5. Follow labeled mixing and application directions. A pesticide should be thoroughly mixed in the proper mixing equipment in the proper proportions with the recommended diluents and applied at the proper dosage

rates with appropriate application equipment. Assure that mixing and application equipment is properly adjusted and calibrated before mixing and application occurs.

6. Assure that applicators or those supervising applicators are properly trained and certified to apply the intended pesticide or pesticides. Be sure there is adequate supervision and followup of the entire mixing and application process.

7. Observe precautions on the label pertaining to the safety of mixers and applicators. If the label prescribes a certain type of respirator and special protective clothing for the mixer and/or applicator, be certain they are available and worn by them.

8. If you have doubts or reservations concerning the use of any particular pest control method, consult with pest control specialists in the Bureau, extension service, or any other group, agency, or person qualified to provide professional assistance. They may be able to help you achieve your pest control goals in an effective and efficient manner and spare you some legal problems and some time and money.

Aquatic Weed Control

Aquatic weeds constitute a worldwide problem that is becoming more critical because of increased use and development of lakes, manmade ponds, rivers, and irrigation channels. Such weeds invade aquatic habitats used by man for irrigation, transportation, recreation, drinking, and other public purposes. They clog drainage ditches, interfere with delivery of irrigation water, cause seepage and evaporation losses, contribute to the failure of canals and collection of sediment, limit fishing and boating in recreational waters, degrade fish habitat, and provide breeding places for biting and undesirable insects. Each weed control discipline has problems peculiar to its own operation. This is particularly true of aquatic weed control since almost every body of water has multiple uses. The usually simple decision of whether or not to manage mechanically or treat with herbicides becomes extremely complex when the movement of multiple-use waters is a factor.

Because the aquatic situation is fluid, mechanically removed vegetation or applied herbicides will seldom remain where placed. This is the essential difference between aquatic and land weed control and one of the reasons some aquatic herbicides may be placed in a restricted use category by regulatory agencies.

Aquatic weed control is further complicated because there is rarely absolute ownership of a body of water. Multiple interests may be considered.

In some instances, approval is necessary from public health departments and fish and wildlife agencies. Eventually we will no longer apply herbicides or employ other measures in terms of only a single aquatic weed problem. Instead, we will manage the aquatic environment with an understanding of the interaction of all steps taken and will likely utilize methods that integrate mechanical, chemical, biological, and environmental manipulation methods together.

Submersed Aquatic Weeds

Submersed aquatic weeds which are rooted to the bottoms of canals, laterals, or drains are referred to as submersed weeds. They grow freely in the water, buoyed by air cells in the leaves and stems. While the word "moss" is sometimes used by irrigation people to identify these weeds, most of the kinds found are not "moss" but are a variety of flowering plants, some of which are called pondweeds (see fig. 5).

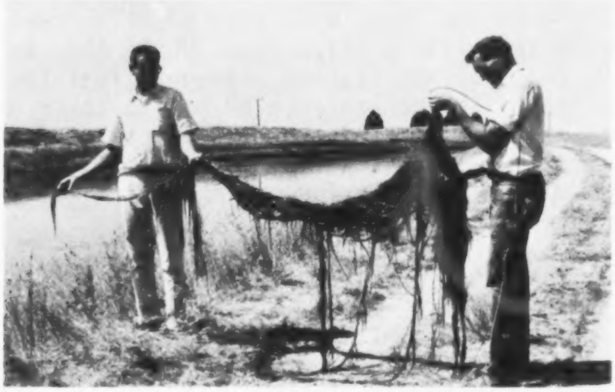

Figure 5. - Sago pondweed, 12 to 15 feet in length, has been taken from a canal on the Columbia Basin Project, Washington.

Mechanical Methods Such as Chaining, Disking, or Hand Cutting

Until 1947 when aromatic solvents were found to be effective in controlling pondweeds, chaining, disking, hand cutting, or other mechanical methods were the major means of removing these weeds from irrigation areas. Mechanical methods are very expensive and create many problems downstream in plugging structures. These methods have been replaced in most areas by the use of chemicals. Recent stringent labeling laws have caused some operators to reconsider mechanical methods over chemicals.

Chemical Control

There are a very limited number of aquatic herbicides that are labeled for use on irrigation systems. These require specific knowledge to obtain maximum effectiveness and to prevent toxic effects on nontarget species. Details on these application procedures are reviewed in the following sections:

Xylene Use for Aquatic Weed Control

Xylene applied with a suitable emulsifier every 3 to 6 weeks (four to six treatments per season), in concentrations of 8 to 10 gallons per cubic foot per second, will control pondweeds and algae in laterals carrying as much as 200 to 300 cubic feet per second.

Effective distance of each treatment varies from 3 to 12 miles, depending on the velocity of the water, density and age of weeds, silt carried by the water, and temperature of the water. Booster treatments of 5 gallons per cubic foot per second are applied as the white blanket turns to gray or blue, indicating that the emulsion has broken and the concentration of xylene needs strengthening.

More positive control of pondweeds and less siltation are possible through the use of xylene applied by the calendar at regular intervals before the plant becomes large and mature.

Irrigation districts and canal companies have demonstrated that 4 gallons of xylene per cubic foot per second applied every 2 weeks, beginning when the pondweeds are 4 to 6 inches long, will give positive control of pondweed and algae with very little siltation. While this method may slightly increase the costs of application, managers feel that the added cost is more than offset by the convenience of having their ditches free of pondweed and algae at all times and the reduced costs of removing silt settled by the weed growth. This method of control is also highly desirable on systems delivering water for sprinkler irrigation since very little debris is produced.

Acrolein Applications for Aquatic Weed Control

This herbicide, a labeled product of the Magna Corporation, is an effective herbicide for use in irrigation canals. It destroys only the stems and leaves of aquatic plants.

Growth Suppression [1] by Acrolein Applications

Experience by irrigation districts and canal companies has proven acrolein to be effective in the suppression of pondweed and algae in large canals (75 cubic feet per second or larger) when applied at a quantity as low at 0.166 gallon per cubic foot per second. The concentration of acrolein required is dependent upon the conditions within the channel being treated. The treatment is most effective under conditions where the pondweeds are not more than 6 inches long, velocities are 1 mile or more per hour, and the water temperature is above 60 °F.

As a general rule, pondweed growth reduction in small streams requires a higher concentration of acrolein than in larger streams. Small channels have more wetted area than larger streams in relation to the total quantity of water carried, and as a rule, the velocity of the water in the smaller channels is slower and the density of aquatic plant infestations is greater. These conditions require higher concentrations of acrolein.

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[1] The term suppression was coined to refer to treatment of weed growth in an early vegetative stage to prevent or reduce its growth vigor before it becomes a problem.

Each canal, lateral, or system of laterals must be classified as to the varieties of pondweed growing in the system, intensity of pondweed infestations, stage of growth or maturity of the plants, temperature of the water, velocity of the flow, miles of weed control needed, and other factors having a bearing upon the concentration of acrolein required. The success of any program of pondweed control can be measured by the thoroughness and accuracy of the manager's or watermaster's knowledge of the conditions in the system, all of which have a bearing upon the timing of applications and the concentration of acrolein required for each canal, branch, or lateral within the system.

Applications Procedures

Acrolein is introduced at intervals of 2 to 3 weeks into the flowing water by gravity through polyethylene tubing. Flowmeters, orifice plates, hose valves, and other application equipment are available. Constant pressure in the acrolein cylinder is maintained by the introduction of nitrogen gas. Oxygen should never be used as it causes the acrolein to polymerize (change to another form with different properties). See figures 6 and 7.

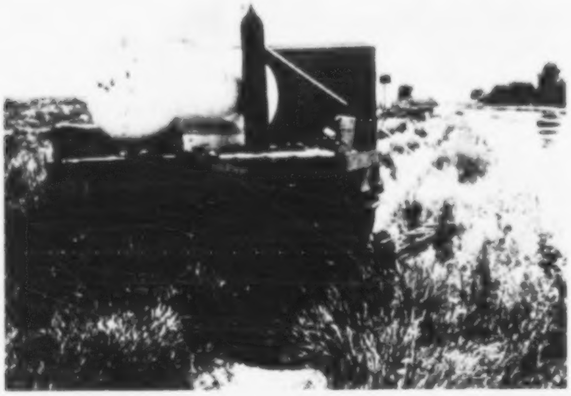

Figure 6. - Small quantity of acrolein being applied through the small tube will suppress the pondweeds in this large (1,400 ft^3/s) canal - 48-hour treatments at intervals of 2 to 3 weeks do the job. Field equipment applying acrolein to Gravity Canal, Burley Irrigation District, Idaho.

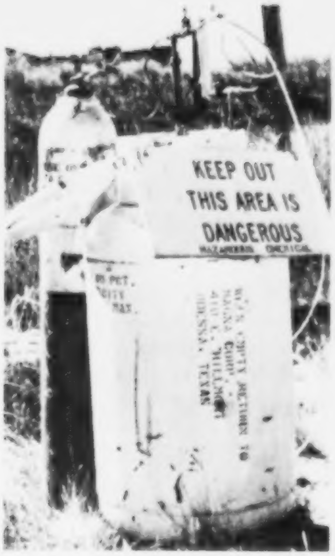

Figure 7. - Closeup of acrolein tank (350 gal).
nitrogen tank, flowmeter, and related equipment.

Treat When Plants are Small

Experience has demonstrated the need for beginning treatment when pondweeds are 3 to 6 inches in length to effectively prevent development of problem weed growth. Delayed treatments, when the plants are too large, may result in some loss of effectiveness of lower rate acrolein concentrations. To be sure that the plants are not too large at the outset, the initial treatment should be made when any pondweed growth can be found. The first symptoms of herbicidal injury are changes in color of the leaves from green to reddish brown, overcast onto green, followed by a light brown color coupled with a "bruised green" appearance. Leaves become flaccid and tend to break away from the stem easily. Stems become weakened, taken on a bleached appearance, and develop necrotic spots.

Low-rate Copper Sulfate Algaecide and Herbicide Applications

Copper sulfate has been used for many years to control filamentous, green algae on irrigation systems. It is applied at a rate ranging from 1/4 to 2 pounds per cubic foot per second of flow as the algae becomes a problem. This application rate is influenced by crystal size, feeding rate, water quality, and water temperature. Care must be taken not to exceed the established water tolerance of 1 part per million for potable water and waters containing desirable fish (see figs. 8 and 9). This technique of copper sulfate application where the dry crystals are dumped into the water of dissolved from bags suspended in the flowing water is generally referred to as the slug method. Although this slug method has proven satisfactory, equipment for controlled feeding of dry copper sulfate warranted evaluation to determine if improvements could be made in the application technique. The availability of good, dry chemical feeding equipment also made it possible to evaluate the effectiveness of copper sulfate for controlling rooted submerged aquatic weeds in irrigation canals.

Studies conducted by the cooperative Bureau of Reclamation and Fish and Wildlife Service aquatic weed research program demonstrated that daily applications of low concentrations of copper sulfate started early in the season can be effective in providing good control of pondweed as well as algae. Results of some of these experiments conducted in various canals in the Western United States indicated that copper sulface feed rates ranging from 1 to 3 pounds per cubic foot per second per day often provided acceptable pondweed control. However, results vary in different geographical areas. Control can be obtained from 5 to 10 miles downstream from one feeding station. Algae control often can be obtained at much lower rates down to as low as one-tenth pound per cubic foot per second per day. Feeding must be started early in the season and continued as long as control is needed.

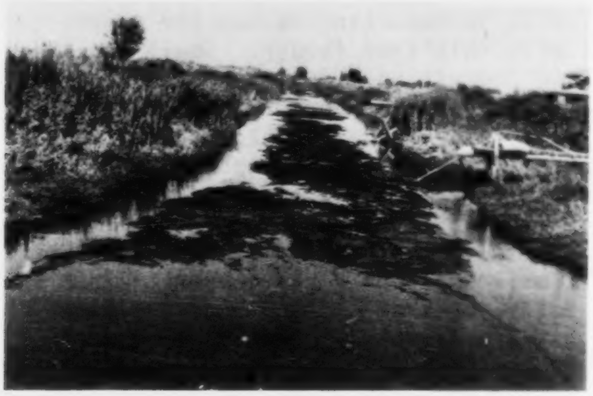

Figure 8. - Algae can be controlled with copper sulfate. View to the east where herbicide was not used, Minidoka Irrigation District, Idaho.

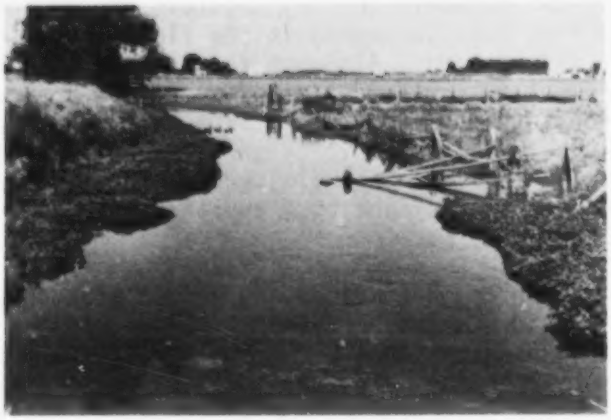

Figure 9. - View to the west of the canal pictured in figure 8. This section was treated regularly with copper sulfate, Minidoka Irrigation District, Idaho.

Equipment, Materials, and Other Factors

The following should be considered if this low-rate method of algae and pondweed control is used:

Copper Sulfate. - It is important to use the proper particle-size copper sulfate crystal with this type feeder. Small crystals in the 1/16- to 3/16-inch size and containing a minimum of dust-size particles are recommended. The material is kept dry by locating the feeder inside a small house or room. The feeder should be located over or near the water so that the copper sulfate crystals can be dropped from the discharge spout directly into the flowing water.

Water Quality. - Alkalinity is an important aspect of the water to be treated since it will react with the copper to form an insoluble copper compound. Generally, as the alkalinity of the water increases the quantity of copper precipitated increases also. Irrigation water in the Western United States on the average is not excessively high in alkalinity.

Rates for Algae Control. - Feed copper sulfate to the water on a continuous or intermittent basis daily at a rate of about one-tenth pound per cubic foot per second per day. Start feeding early in the season and continue throughout the season or as long as control is desired.

Rates for Pondweed Control. - Feed copper sulfate to the water on a continuous or intermittent basis daily according to the following schedule:

3 pounds per cubic foot per second per day for the first one-third period of season
2 pounds per cubic foot per second per day for second one-third period of season
1 pound per cubic foot per second per day for third period of season

The above recommendations for algae and pondweed control are suggested as guidelines for the average situation. Factors such as alkalinity of water, length of canal, and density and kind of weed problem would influence an increase or decrease in the above-suggested feeding schedules. Each canal situation have to be evaluated separately to obtain an optimum application procedure.

Feeder. - Volumetric screw-type feeder with hopper for dispensing dry copper sulfate crystals and equipped with controls to provide a constant or variable rate feed - the constant rate feeder equipped with a time clock to operate feeder for predetermined periods of time is suggested for use on canals where water discharge is approximately constant for at least a few days. The variable rate feeder is recommended for use on the canals where water discharge fluctuates considerably over short periods of time. In addition, certain less complex feeding devices have been incorporated into effective copper feeding techniques with some success. Simple devices to hold and slowly release the chemical by dissolution can be adapted. However, they do require considerable attention and have limitations on wide ranges of feed control.

Invertebrate Animal Growths in Closed Systems

This discussion is offered as a brief introduction to familiarize you with those organisms that may be found growing in closed systems as opposed to open systems. Comparatively little has been published on the biological growths in aqueducts and pipes. These growths can be particularly important to you from a hydraulic aspect in the operation of the system.

Attached algae and higher aquatic plants are the common biological pests in open canals and flumes and require light for survival while animals, such as sponges, bryozoans, mollusks, fungal plants, and bacteria are able to exist in the darkness of closed systems. These animals grow in conduits and can survive without light.

Most of the animals found in closed systems are classified as invertebrate animals. Familiarity with their gross features will aid in a tentative identification. Since many of the characteristics used in identification are very small, a detailed microscopic examination is necessary for positive identification of genus and species. However, precise knowledge of the classical identification of these growths is not necessary as their general appearance differs significantly between types. A general description of the organisms and the problems they cause in closed systems are offered as follows:

Fresh Water Sponges

These are simple animals with bodies made of unspecialized tissue without specific organs. They consist of interconnected channels, chambers, and orifices. The support skeleton of the animal is made up of random groups of needle-like bodies called spicules surrounded by gelatinous masses and a horny fiber called spongin. This is the material that makes up the body of common commercial marine sponge, a familiar household item.

The animal often goes unnoticed because of its inconspicuous growth, character, and color. Oftentimes the animals are brown or yellowish in colorations but sometimes greenish. However, they may become extensive, covering large areas and grow up to 1 or 2 inches thick. Generally, you would view these as mat-like encrustations with a lobed form of growth. The one most easily recognized character differentiating this animal from other mat growths is the support skeleton or spicules. When mature sponges are handled, they feel somewhat rough or slightly granular. The spicules have been known to cause skin irritation when handled and eye injuries have resulted.

The animal prefers dark locations, attaching to any suitable submerged objects where a suitable environment exists, such as closed conduits.

The sponge food source is microscopic organisms and detritus that are carried to the organisms by flowing or drifting water. Generally, the major food source is minute plankton that grows in open, lighted situations and is carried into the closed conduits where the animal thrives.

The animal survives adverse conditions during dry periods and is disseminated by a small whitish structure called a gemmule. These are often seen in dewatered structures where the sponge has been growing.

Sponge growths have been observed in both open and closed irrigation systems (see fig. 10). However, the major problems reportedly associated with these animals have been on open, lined canals on the Columbia Basin Project in Washington and the Central Valley Project in California. Inspections of closed conduits on the Charles Hansen Feeder Canal of the Colorado-Big Thompson Project have revealed sponge growths up to 1-1/2 inches in thickness on the concrete tunnel walls. The area covered by these growths has been only a small portion of the total surface area. A number of reports have been published in technical literature indicating serious plugging of municipal water conduits by sponge growths. However, to our knowledge they have been only relatively minor problems on irrigation closed conduit system.

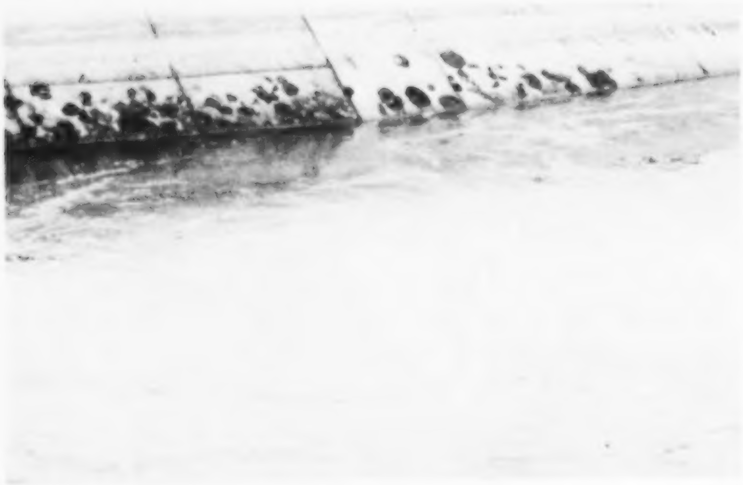

Figure 10. - Freshwater sponge growing on the concrete lining of the Delta-Mendota Canal, California.

Bryozoa

These animals are sometimes referred to as pipe moss and often appear as a brownish mass resembling dead moss or plant material. They grow on submerged objects in relatively dim light to total darkness. The animal itself is microscopic, more or less cylindrical, and branched. The growth and proliferation of the bryozoan colony is by asexual budding, creating large mats of masses covering entire wetted perimeter of open, lined canals or closed conduits. An illustration of bryozoa is shown in figure 11. Figure 12 shows a complex bryozoan growth on a concrete canal lining in California.

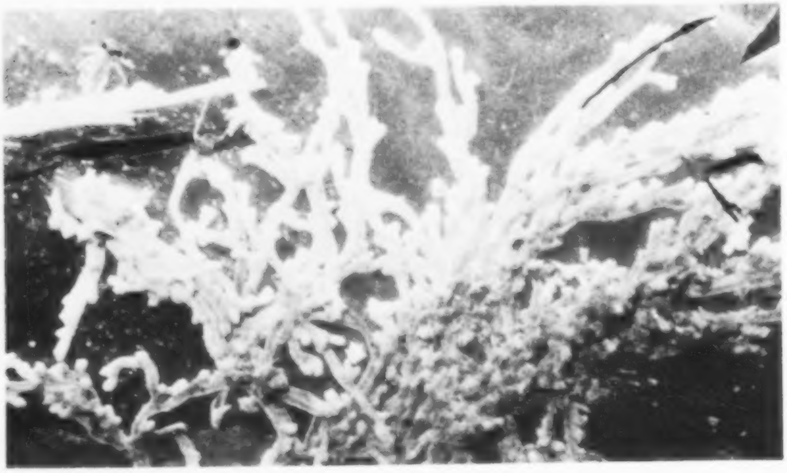

Figure 11. - Bryozoan colony growing on a glass plate that was immersed in a canal in Colorado.

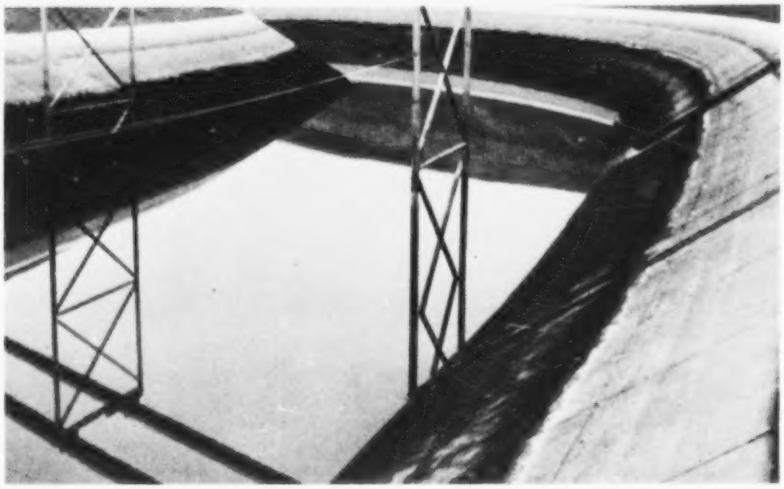

Figure 12. - Bryozoan growth on concrete canal lining.

A unique feature of the bryozoa is their ability to produce a seed-like body that is highly resistant to drying or cold temperatures. These are classed statoblasts and provide for the overwintering and dissemination of the organisms in aquatic habitat.

Bryozoans, like sponges, depend on microorganisms and detritus suspended in the water as a source of food. These animals are seldom found in polluted water and require high dissolved oxygen concentrations.

Bryozoa have been found growing in a number of situations on irrigation projects such as pipelines carrying irrigation water through the city of Kennewick, Washington; the louvers of the Tracy fish collecting structure near Tracy, California; and tunnel walls of the Colorado-Big Thompson Project, Colorado. Technical literature reports of bryozoan plugging of municipal-industrial water systems are not uncommon.

Mollusks

The bivalve mollusks found on irrigation systems are commonly referred to as clams or mussels. The Asiatic clam is the one that has been found in irrigation canals of Arizona, California, Oregon, Washington, and Idaho, and the major rivers of this area such as the Colorado, Snake, Columbia, and Sacramento. Another clam that has caused a lot of pipe plugging in Europe is commonly called the zebra mussel. Fortunately, this clam has not been found in the United States.

The Asiatic clam seeks areas where it can burrow into the bottom sediment and feed on plankton and other food carried to it in the water. An open system is, therefore, a more favorable habitat for its growth and development (see figs. 13 and 14). Since in the larval stage the clam is very small and soft bodied, it is readily transported by water, and closed systems can be infested with the immature clams. If these larvae find a suitable environment in the closed system, they will grow to maturity, resulting in significant loss in flow capacity of the structure. In some cases, the adult clam may be flushed into a closed system. The clam may not survive under these conditions, but the shells may become a problem by clogging certain structures or by forming a deposit of shells if they are not flushed through the system.

We know of one instance where the clams have flourished in a closed system. This occurred in the cooling water system of the synchronous condenser at the Tracy Switchyard, Tracy, California. The clams grew and plugged the pipes of the cooling system. These have also been reported as serious problems in cooling condensers in other geographical areas of the world.

Insects

A number of aquatic insects spend their immature life stages in freshwater. In one or more stages (larval and pupal stages) of their development, they may attach to rocks, logs, or masonry structures. Numerous insect larva could cause some loss of capacity in pipelines and other closed structures.

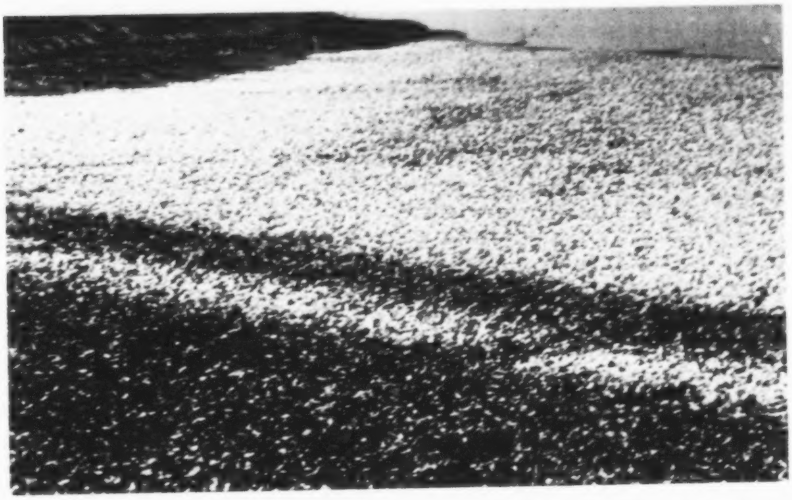

Figure 13. - Large bar of Asiatic clams and silt that developed in the Delta-Mendota Canal, California.

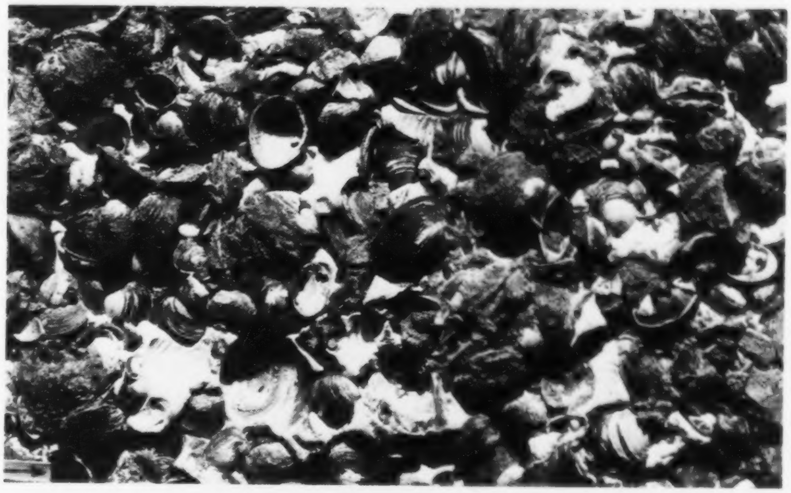

Figure 14. - Asiatic clamshells that were collected from the Delta-Mendota Canal.

Dobsonflies, caddisflies, and trueflies are groups that have been found to infest pipelines. The Southern California Electric Company has reported that larvae of the caddis and dobson flies have caused up to 8 percent reduction in the capacity of water conduits in one of their plants. Caddisfly larvae infested the conduits of a powerplant in Nebraska. The pupal stage of the blackfly has been known to attach to irrigation structures in Idaho and other areas. Growths of blackfly larval encasements growing on a rock substrate are shown in figure 15. They oftentimes develop on concrete canal linings and conduits in a similar fashion.

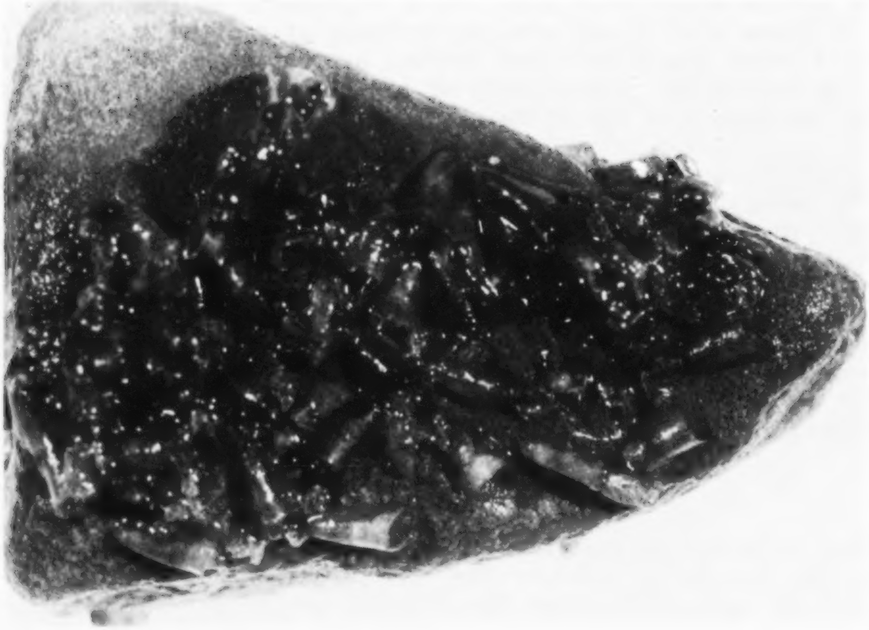

Figure 15. - Blackfly insect larval encasements growing on a submersed rock in a canal.

Other Aquatic Organisms

A number of other organisms including fungi, worms, crustacea, protozoa, and rotifers are often found on the walls of closed systems. These types of organisms, many of them microscopic in size, are often found growing and living in an association that has a brown to gray color and a mat-like appearance. An encrustration of this type ranging from 1/8 to 1/4 inch thick was collected from a tunnel on the Colorado-Big Thompson Project recently and examined. It was found to contain organisms belonging to the aforementioned groups. The dried encrustration was analyzed and found to contain 8.8 percent ferric oxide and 8.7 percent manganic oxide. Copper, sodium, calcium, aluminum, magnesium, and silica were also found in the dried sample. The filamentous bacteria observed in the sample were probably responsible for the high content of iron and manganese found in the encrusted material. Certain bacteria oxidize these elements in utilization as an energy source, resulting in the formation of insoluble compounds deposited in the encrustation.

Effects of Pipe Growths on Closed Systems

The major problems the organisms described above are capable of creating in a closed system for delivering irrigation or municipal-industrial water in multiple-use water systems are the reduction of the capacity of the conduit and plugging of various structures, particularly small diameter pipes and orifices.

Although the pipe growths have caused relatively few reported or recognized problems on irrigation projects to date, it is reasonable to assume that the problems will increase with an anticipated increase of construction of closed systems. Also, a trend toward enriched surface waters in the form of nutrients will increase the biological productivity of the water which will increase the food supply for the growth of the animals.

The hydraulic problem caused by these growths could be kept to a minimum by considering high-velocity flows, smooth surfaces, and elimination of structures, small pipes, and other facilities prone to clogging in the design of new closed systems.

We suggest that you observe your structures for growths of the types described in this discussion. We will be glad to help in the identification of organisms of this type that you may find and the recommendation of control methods if they are needed. Any growths found should be kept under surveillance to determine whether the infestation will develop into a major problem.

Control of Pipe Growths

There is not much information available concerning the control of the animals described. Various mechanical devices that use scraping and brushing techniques have been used to dislodge the attached organisms. Chlorine is probably the most frequently used chemical agent for control. Emulsified xylene aquatic weed herbicide has proven effective on clams and bryozoans in a few applications. Copper sulfate has also been reported effective on certain of these organisms. In addition, control of plankton food supply by reducing level of nutrient enrichment of water or algaecides might also reduce populations of these animals. Any control method employed will need to be developed and refined and applied on schedule in much the same manner as you use in developing good weed control programs.

Water Systems Operation and Maintenance Workshop - 1990

DRAINAGE OF IRRIGATED LANDS

Sessions 13-1 and 13-2

Keith Campbell and Rod TeKrony

The science and engineering of drainage have come a long way in the last 3 decades. Today drainage is out of the "put the drain here and see how it works" stage where it had so long been. Although sound judgment is still essential to a good job, the technical tools are not available to make drainage an engineering and economically feasible undertaking. This is fortunate because, except in rare instances, drainage is essential to sustained high productivity under irrigation. This is the way nature made it, and man cannot change it.

Irrigated areas start with certain basic characteristics. There are given soils underlain by various substrata and geologic formations, and the area has a certain topography. There will be a wild plant community of some sort. Rain and snow fall more or less regularly, and under nature all these things have reached a balance. They stay that way until farmers come along. They clear the land, level it, irrigate it, plant crops, try for high production, and put on more water than native crops required. What happens? Waterlogging or salinization, or both - they usually go hand in hand. These are drainage problems, which lead to a simple definition for drainage. It is the "removal of excess water and excess salt from agricultural lands."

Most of our discussions will deal with the complexities of drainage that are concerned with what happens beneath the surface - subsurface drainage. Surface drainage is important, but it is a natural process. Water runs downhill; it must be kept moving, not allowed to pond, so the land will not get too wet from it. This is surface drainage. The other kind of drainage that is so essential, subsurface drainage, is a form of land maintenance. Dams, canals, pipelines, and roads must be maintained; land must be maintained also. Drainage is an important form of maintenance. With enough good drainage, even the poorest farmer cannot very well ruin land. With poor or not enough drainage, the best farmer cannot be successful. Sooner or later salts or ground water, or both, will rise to the zone of the soil where the plants grow. This may take 2 years or 20 years, sometimes longer.

Irrigation or drainage districts are basically concerned with the drainage of irrigated lands, and sometimes with lands that are adversely impacted because of nearby irrigation. The principal objectives of districts should be to predict and prevent waterlogging and to keep salts and alkalies away from the plants. They should try to avoid the necessity of having to reclaim waterlogged lands, by not allowing them to become that way in the first place.

The general feeling among people not closely associated with drainage problems has been that the problems do not start for a good many years after irrigation. Although there are exceptions, in most cases this has not been true. The problems may sometimes develop slowly, but they begin soon after irrigation starts. Sometimes the causes are not readily apparent, and careful analysis is needed to locate them. Often new problems develop merely because of changes in cropping practices, like changing from alfalfa to beans. Sometimes new problems develop when farms change hands. Usually, however, assuming reasonably good farming operations and good water management by the irrigation district, the problems stem from basic natural land characteristics or faulty irrigation works.

Inadequate subsurface drainage adversely affects plants in two ways. One way, the root zone becomes waterlogged and the plants literally drown. They cannot get needed oxygen. The other way, the plants are poisoned or starved of nutrients and moisture by excessive salinity in the root zone. As the deep percolation from irrigation moves downward, it dissolves some of the residual salts in the soil. Also, the salinity of the moisture within the root zone is concentrated because the plants basically remove pure water and leave the dissolved salts behind. When this water reaches the ground-water body, it adds this salt load and the ground water becomes increasingly saline. More deep percolation results from each irrigation, and gradually the water table builds up toward the ground surface. Then the phenomenon of capillarity becomes an important factor in the salinity problem.

Capillarity is the force that moves oil up into the wick of a lamp, against gravity, or ink into a blotter. Most people have seen some form of capillarity in a science laboratory. Perhaps they suspended a capillary tube in a liquid. The liquid rose in the tube above the level of the liquid, and the smaller the diameter of the capillary, the higher the water rose. The same thing happens in soil. If the ground water is too near the surface, capillarity starts working and the direction of water movement is upward. As the plants take up and use the good water from irrigation stored in the capillary pores of the soil in the root zone, the salty ground water moves upward through the chain of small capillary pore spaces in the soil to replace the good water. If displacement of the saltwater downward by good water from irrigation or rain does not occur soon enough, the salty water eventually reaches the surface or near enough to it to evaporate. When this salty water evaporates, which is very quickly in the arid West, the salt remains and rapidly accumulates. After this happens, no plants can grow. Due to capillary rise of water in soils, excessive salinity can and often does occur in the root zone even when there is not waterlogging of the root zone.

These are the things that must be controlled, and only by proper management of irrigation and adequate drainage can they be controlled. The slat must be kept out of the root zone by irrigation with the application of more water than the plant can consume, and the ground water must be kept below the depth that permits saturation of waterlogging of the root zone. There is complete failure of agriculture when the salts become excessive or the ground water becomes too high. Complete failure is a sad sight, and often severe sociological problems develop. Farmers abandon their farms after having lost all they had invested, to become relief and resettlement problems; merchants fail; and settled communities disappear. History of past civilizations indicate that waterlogging and salinity have caused entire civilizations to disappear.

Most western crops have deep roots because those with shallow roots generally have not been able to produce the income required to pay the cost of irrigation. An exception would be in areas with long growing seasons where double cropping of shallow-rooted vegetable crops is possible and good markets exist. Deep-rooted crops need a well-drained zone about 4 feet deep or more. This requires that the noncapillary, or gravity, water will move out of the root zone in about 24 hours after irrigation so the roots can get air. If this zone is saturated longer, the plants begin to suffer; if saturated too long, they die. In planning a drain system, consideration must be given to the height to which ground water can be allowed to rise and saturate the soil and the required rate of removal of the excess ground water. This rate will depend on the hydraulic conductivity, or permeability, of the soil and on the storage capacity available below the roots to receive this water and store it as it gradually lowers and moves into drains. There must be enough drainage to allow the storage space to be available again for the next deep percolation event. Storage capacity of a soil is measured by its specific yield characteristic, which is the amount of noncapillary pore space in the soil that will hold water and allow it to run out by gravity, if it has a place to go. If it has no place to go, the soil is saturated or waterlogged. Specific yield and permeability are two important soil characteristics to be measured during an investigation for designing and building drains. These factors plus depth to barrier and the amount of deep percolation to be handled largely determine drain spacing and depth which, in turn, determine the costs of drainage.

Although some people of the irrigated West have been talking drainage for over 50 years, most of the real understanding of the pertinent phenomena has come in the last 30 years or so; the analytical capability has come mostly in the last 25 years. The earlier effective work came about through a practical approach of cut and try, which always results in some waste and some unsuccessful work. Probably the principal factors that generated the recent progress were signs of mounting problems and a real desire of the farmers and irrigation districts to face and resolve them in view of the comparative agricultural and national prosperity.

Those really interested in drainage and responsible for getting it done economically had to start practically from scratch in building a sound technical approach to sure-fire handling of drainage problems. It was soon learned that the provision of adequate drainage at least cost, or the accurate prediction of the drainage and money that would be required to protect lands that were proposed to be irrigated, demands a study of the problem and the collection of data - in other words, an investigation. This is because the irrigated valleys of the West usually present a complex relationship of soils, water, crops, and irrigation practices which must be evaluated to varying degrees in the economical solution of drainage problems. Some drainage problems and, hence, some drainage investigations are much less complex than others. Although investigations should be kept to the minimum commensurate with satisfactory results, that part of the drainage dollar spent on investigations is usually well spent, and very seldom, if ever, can economies in drainage construction be achieved through skimping on investigations.

A key factor in drainage or in drainage investigations is the water table. Irrigators, irrigation district management, agriculturists, and engineers orient much of their work around the water table. Successful irrigated agriculture in an arid environment requires good control of the water table. The basic questions to be answered in drainage investigations usually boil down to these: Where is the present water table? Is it rising or static? Where can it be expected to level off with continued irrigation? Is the water table too high or will it be too high when it reaches equilibrium? What manmade works are required to control it at safe levels? What will be the cost?

One of the first steps in a drainage investigation is to determine the source of the ground water causing the problem. The source must be known in order that the proper protective measures can be undertaken. As a first step, local irrigation practices should be observed and the amount and distribution of their irrigation should be discussed with farmers. Attempts should be made to determine whether the problem is or may be expected to be general for the whole area or whether it is applicable only to specific portions of the area. Perhaps the area has always had poor natural drainage and the drainage problem is merely the result of encroachment of irrigation upon natural conditions. The problem may have developed from construction of a dam or a leaky canal or it may have developed from changes in irrigation practices. Sometimes it may be the result of cycles of high rainfall, the expansion of irrigated areas, or of periods when excessive irrigation water has been applied.

At least some investigational data are normally needed to go further into the problem and pinpoint the source of the ground water. Test borings and observation wells will indicate ground-water levels, directions of movement, fluctuations, and possible causes. Some in-place permeability tests are usually essential. In addition to the permeability of the principal strata data are needed on the type of soil and thickness, position, and continuity of such strata. It has been learned, sometimes the hard way, that no matter how favorable the soil and subsoil characteristics of an area appear to be, at least some knowledge of deeper substrata is also required.

There are only a few sources where ground water can come from, and it is usually not too difficult to pinpoint the source causing the trouble. Even in the Western States, rain can sometimes be the cause of the high water table, so precipitation records should be compared with ground-water levels. To determine whether excess irrigation is the source of the drainage problem requires a study of the effect of individual irrigations on the water table, the fluctuations of the water table throughout the irrigation season and during times of no irrigation, and changes in water-table elevations over a period of years, both before and after the beginning of irrigation.

In many drainage problem areas, seepage is a major source of the ground water. Most seepage comes from canals, laterals, reservoirs, or the irrigation of higher lying lands. Here again, comparisons are made of ground-water fluctuations with water levels in, or the presence of water in, canals or reservoirs or with irrigation at higher levels. Sometimes cattails, willows, or other water-loving plants indicate seepage and a possible source of recharge. Other methods of detecting seepage involve the use of dyes, salts, and piezometers. Lately, radioactive tracers have been used with some success.

Artesian pressure is sometimes a source of high water tables. In this case, piezometers are needed to define the problem and solution. Artesian pressures are usually found where a slowly permeable layer overlies a saturated permeable layer whose source of ground-water intake is at a higher elevation. Water may be forced upward into the root zone, through the slowly permeable layer or through fractures, displacements, or permeable lenses in this layer. Sometimes old artesian wells leak below the ground surface or are allowed to run freely without proper facilities to dispose of the surface flow. Drainage problems under artesian conditions are usually difficult to pinpoint and more costly to solve.

Once the source of the high ground water is determined, a start has been made on the solution to the drainage problem, and it will then be known what additional information will have to be obtained. This would usually include more data on ground-water levels and fluctuations; salt concentrations in both water and soil; past and probable future irrigation and cropping practices to permit evaluation of irrigation schedules and deep percolation; quantitative losses from irrigation channels; and permeabilities of soils, subsoils, and the stratification thereof. More or less standard techniques have been developed for obtaining this information, but skill and good judgment are required to get reliable answers.

After the investigations are completed, what can be done about removing the excess water and the salt? The ground-water table can be lowered and the salt can be moved by leaching it. In leaching, still more water is put on to dissolve the salt in the soil and move it to safe depths. But more water cannot be put on until a place is provided to put it, because the ground water would only be raised still more and then all the problems would be intensified. Drainage is the solution.

There are several kinds of drains. Open drains are simply open ditches excavated to the right depths for the local conditions. Closed drains are constructed by burying pipe or plastic tubing in the water-bearing strata. Closed drains have the great advantages of eliminating weed and silt problems and of permitting cultivation over the drain. There are also other kinds of drains which are not so well known. Pumped wells are one. The Salt River and Wellton-Mohawk Valleys in Arizona, as examples, are drained by pumped wells. These wells remove ground water, which is the objective of drainage. Sometimes the water can be reused by mixing with canal water and sometimes it cannot because of the water quality.

Another method of drainage that many people are not familiar with is by inverted or recharge wells. These are upside-down wells. Instead of taking water out of the ground, they are used to put water into the ground in places where it will do no damage. Inverted wells serve as drainage outlets. First, there usually is some kind of a conventional drainage system that gathers the water and takes it out of the root zone. Then, where no natural or cheaper outlet is available, wells are drilled into unconfined aquifers that can take the water conveyed to them from the drains. There are examples of inverted well drainage in Idaho and in Texas. In the Snake River country of Idaho some of the underlying basalts are fractured, and the water that gets into this material through inverted wells eventually finds its way to the river through subsurface channels. Recent environmental restrictions must be followed when considering the use of inverted wells.

In recent years, disposal of drain system flows have become a problem in some parts of the country. The quality and estimated quantity, including specific constituents of drain flows will need to be determined before a disposal method can be selected. For most locations, mixing with existing surface flows will be adequate. Local and state regulations should be checked before a disposal method is selected. Some states require onsite disposal when base quantities of certain elements are exceeded. Remember that maintaining salt balance in the soil profile is a primary goal of drainage. Any plan which does not meet the salt balance goal is not a long-term solution to the drainage problem.

Up to about 1972, pipe for closed drains were made with clay or concrete, but now most subsurface drains are constructed with corrugated plastic drain tubing. Specifications have been developed for the plastic drain tubing which include requirements that assure a long life for the tubing when installed properly. Irrigation districts should try to develop the best drain designs and use the best material for the purpose so their farmers get the most for their money over the long haul. To do this, the drains sometimes require more initial investment in materials, in methods of placement, and in quality of design than would shorter-lived structures.

Most drainage specialists of this country are convinced that closed subsurface drains are much better than open ones, except where capacity requirements are so great that the size of closed drains would be prohibitive in cost or where considerable surface waste or storm runoff must be handled. Pump well drainage and inverted well drainage are used only in special instances where aquifer conditions are ideal and the costs are less than for open or closed drains. Feasible pump well drainage requires low-cost electric power and chemical characteristics of the soil and water that permit long life of the well components.

In most localities, costs of closed drains are less than for open drains of the same depth if all factors are considered. Maintenance costs of properly designed closed drains are insignificant compared with those for open drains, and the productivity value of the land taken out of cultivation by the open drain is appreciable, whereas all the land over closed drains can be cultivated.

The cost of drainage is an important economic factor, deserving of serious thought. Although there are many miles of drains already built on irrigation projects, this is only a fair start on the total needed for optimum productivity of irrigated western lands. Economic fluctuations have had some effect on rate of drain installation, more so in the humid areas, however, than in the irrigated arid lands. Drainage in the West, because of the salinity and seepage factors, is so often vital to good productivity, even for pasture. Since farmers, as well as public and private financial assistance agencies, now know much more about the benefits of drainage than they did 25 years ago, progress will doubtless continue year after year, even through economic depressions.

While there are some exceptions, today's costs for providing adequate drainage for irrigated lands range from about $100 to over $1,000 per acre. This is a high price for most farmers to accept and finance, even though they can see the ultimate benefits. Most of these costs are in the material and the installation. The improvements in the use of plastic tubing have been much greater than the advances in equipment well suited to construction of good closed drains. Few contractors are equipped to install efficiently, for example, a first-class, 10-foot-deep closed drain. Most drainage requirements are progressive - an area here this year, one there next year - and therefore contracts are usually small compared with those for construction of other phases of irrigation projects. The work being small, scattered, and usually spread over several decades, is not sufficiently attractive to substantial contractors who would have to buy drain trenching equipment for the job. This results in little competition, limited generally to small, local contractors, and high costs. Many contractors are equipped only with draglines, backhoes, and similar equipment, which usually cannot perform efficiently on drainage work, but a few are now buying trenchers capable of excavating up to 12 feet.

Drainage contractors in irrigated areas of California and Texas have developed specialized pipe and tube laying trenching machines which do a good job at minimum cost. The most versatile of these rigs will dig trenches up to 12 feet deep; place or permit easy and safe placing of the pipe or tubing in the trench; and place gravel envelope under, around, and over the pipe; all without caving. Some also backfill the trenches over the pipe. Where such machines are locally available, contract costs are often much less than where the contractors have only conventional excavating equipment. It would undoubtedly be wise for some irrigation districts to invest in such highly efficient, specialized drain construction equipment.

Through the years, various formulas have been used for determining drainage requirements such as depth, spacings, and discharge capacities. These formulas have been based mostly on empirical concepts and opinions or experiences of individuals, rather than on more exact engineering methods. With luck, the drainage job or estimate came out about right. Frequently, however, the guesses were too far off, or the empirical relations were not applicable to the particular case. As a result, many irrigation districts suffered in various ways. In recent years, substantial technical breakthroughs have been made in this regard.

Ways have been developed to use measurable soil and irrigation factors to compute more accurate answers than ever before possible with empirical data and variable judgment. Measurable field data can now be taken on soils and substrata from any place, any amount of recharge from either rain or irrigation can be assumed, and the spacing of drains which will be required to control the ground-water table under any irrigation, operating, or cropping conditions can be accurately estimated. Not only can the elevation of the water table be considered, but the time during which it encroaches upon the root zone, the time required for it to decline to a predetermined level, and any seasonal variations can be specified.

These new methods of predicting and determining subsurface drainage requirements have been proved by comparing computed drain spacing and water-table behavior with measured values for areas where drains were already installed. Data on ground-water position and fluctuations, drain discharge measurements, and permeabilities have been used to compute spacings that would be required to give the same amount of water-table drawdown as had been actually measured in the field. In one such study, 67 comparisons were made from such widely separated irrigated areas as Australia, Canada, and Southwestern United States. Computed water-table elevations at specific time periods were within 5 percent of the measured height in 54 percent of the cases, within 10 percent in 79 percent of the cases, and within 15 percent in 88 percent of the cases. Ground-water prognoses, historically, have never before been nearly this accurate, so these results are highly gratifying and confirm the usefulness of the new methods. These results also amply demonstrate the value of in-place permeability tests. Such tests, when field-tested procedures have been followed, undoubtedly provide values of permeability which are realistically close to the actual.

Drain spacing and depth are interdependent and for given conditions there is an economically optimum combination. For some lands 7-foot-deep drains 200 feet apart might be least costly, while in another area 12-foot-deep drains 800 feet apart might be less expensive. Spacings may vary from 50 to 1,500 feet. Nine feet is a practical depth for most closed drains, but the size of pipe, depth, and spacing must be selected on the basis of the physical characteristics of the substrata and the amount of deep percolation to be handled, which depends on the local land use and irrigation practices. Once the proper determinations of location, spacing, and depth have been made, the construction designs for closed drains are simple compared to many other irrigation structures. Even so, several basic elements of design are necessary for an efficient drainage system.

One highly desirable element is a gravel envelope around the drain pipe or tubing. Total amount of drains needed for a required job can sometimes be reduced as much as one-half simply by using the right gravel envelope. Usually, at least 4 inches of gravel should be placed completely around the pipe. This will help stabilize the pipe and prevent movement into the pipe of the soil being drained, and at the same time act as a conductor for movement of water from the soil into the cracks between pipe joints. For plastic drain tubing, the gravel also serves as a support for the flexible tubing. An all-important rule is that permeability of the envelope must be high compared to permeability of the soil, sometimes called the base material. Some pit-run gravels will meet the requirements. The gradation of the envelope should be well graded with nothing larger than 1-1/2 inches and less than 5 percent passing the No. 200 screen.

Other elements of good closed drains are manholes, junction boxes, or silt traps. These are valuable because they provide better inspection of a newly constructed line, easier and cheaper cleaning and repair, and a means of finding blockages and measuring discharges of individual segments of the line. The last operation in constructing an effective closed drain is a careful job of backfilling; to assure that the gravel stays in place and the pipe joints are not moved or broken, and the surface is left as close to original conditions as possible.

All in all, if drainage problems are approached with the knowledge available today; if the designs are properly developed for the conditions; if good construction is insisted upon; if the irrigation district consistently conducts a first-class maintenance and water management program; and if farmers wisely perform their job of good water and land use, then profitable farming operations and financially healthy irrigation districts are assured.

Water Systems Operation and Maintenance Workshop - 1990

BASIC PUMP, MOTOR, AND ELECTRICAL MAINTENANCE

Session 14-1

Earl M. Corless

During this discussion, we will be considering basic or simple electrical and mechanical maintenance, as applied to pumps, motors, and associated equipment. It is my belief that pump and electrical maintenance is really quite simple. A good basic mechanical knowledge and common sense, applied in a timely manner will remedy most situations. We will be discussing pump, motor, and switchgear maintenance; for in reality, a pumping unit consists of all three components. A failure in any one of the three will take the pump out of service.

Generally speaking, electrically driven pumps are extremely reliable. One example of this is that, on the A&B Irrigation District in southern Idaho, the annual budget for maintaining 198 pumps, motors, switchgear, 400 distribution transformers, and substation equipment, is less than the budget for maintaining the district's mobile equipment consisting of 32 pickups, 9 trucks, and 24 pieces of heavy equipment. The fact that electrical equipment is so reliable undoubtedly contributes to failures since it continues to operate in substandard conditions giving us a false sense of security, thereby leading to neglect of proper maintenance procedures.

Pump Maintenance

There are many types, sizes, and brands of pumps. These include vertical turbine, horizontal centrifugal, end suction, and others; also combinations such as booster pumps with deep well pumps. Regardless of the type or size, the major components and maintenance items are similar.

The major maintenance areas are as follows:

1. Lubrication. - This is the most obvious. Always use the proper type of lubricant and in the proper amounts. Too little lubricant will cause excessive wear and lead to failure, whereas too much can also cause failure through excessive heat buildup. Always use top quality lubricants, resist the temptation to economize by using lower grade materials.

2. Packing or "stuffing box." - Pump sleeves with wet lantern-ring packing can be a high maintenance item. This can be significantly reduced by flame spraying the shaft sleeves with stainless steel or other materials and carefully selecting the packing. An example of this is a pumping plant in the Rupert, Idaho area. Prior to changing

the packing system the shaft sleeves had to be removed and repaired annually. The change consisted of metalizing the sleeves with stainless steel, and installing grease-impregnated wood-metal packing. One of the impellers was removed after 16 years of service and the shaft sleeves were only worn from 0.002 to 0.004 inch. The five large pumps in this plant are continuing to operate without significant wear on the sleeves.

3. Vibration. - Excessive vibration, a change or worsening of the normal vibration pattern is always a sign of trouble, and unless corrected, will almost certainly lead to damage or failure.

4. Foreign materials. - The most common problem affecting pumps that pump from surface sources is floating trash and aquatic vegetation. Even small amounts of trash will reduce pump output. Foreign objects such as plastic, sticks, rocks, and even rodents and reptiles are picked up by pumps. Adequate screening should be provided to prevent entry of foreign materials into the pump, eliminating damage to the equipment, and substandard performance.

5. Cavitation. - Cavitation can be caused by improper pump selection or operating the pump outside the range of designed head conditions. A sharp crackling sound coming from the pump while in operation is an indication that cavitation is occurring. This sound which is similar to rocks passing through the pump is frequently heard while filling an enclosed system or when the pump is throttled. When cavitation erosion is also occurring within the pump, very serious damage can be done in a short length of time. Pumps should never be operated over long periods of time while cavitation is going on. In large pumping plants, where feasible, the pumps should be inspected annually. Cavitation and impeller wear will usually start any place within the pump where water abruptly changes direction. Applying coatings to the impeller and pump interior can effectively reduce the wearing process.

6. Low pump discharge. - Perhaps the most common complaint is that the pump is low on discharge. It is essential that you know if the complaint is valid. The past operating history of the pump and motor is invaluable when determining if a pump is not operating properly. In order to obtain this history, measure the quantity of water that is being pumped. Use a good amperage meter or watt-hour meter and measure the amount of power that is being used by the motor. Keep a permanent record of these measurements; then when trouble arises, you will have solid information to assist in troubleshooting, and making the decision to remove and repair the pump. To compare a one-time reading taken during the crises with motor and pump nameplate data is totally inadequate; past operating history is absolutely essential.

It should be remembered that if no other data are available, the amperage or watt-hour meter readings will provide an excellent reference as to the condition of the pump. An electric motor in normal operating condition will use only the amount of electricity that is necessary to operate the pump. If pump efficiency decreases, the power consumption will also decrease.

Almost all cases of low pump discharge are caused by excessive wear increasing the clearance at the pump seal or wearing rings. This is not to be confused with the shaft seal or packing gland. The seal is the interface between the rotating impeller and the pump case. All pumps have one or more seals, and they are usually fitted with replaceable wearing rings. On large pumps, the clearances should be measured and recorded during the annual inspection.

A pump that is low on discharge can be readily repaired at any good machine shop. Satisfactory repairs can be made by: (1) boring out the wear rings, assuming that there is sufficient material remaining, or replacing them if they are badly worn; (2) building up the impeller with stainless steel or other materials; (3) machining the impeller to proper tolerances, and (4) replacing bowl shaft bearings on vertical turbine pumps. Recommended tolerances between the impeller and the wearing rings are for a 6-inch-diameter impeller, 0.014 to 0.016 inch; for an 8-inch-diameter, 0.016 to 0.018 inch; for impellers above 8 inches in diameter, add 0.001 inch for every additional inch of diameter; and for large impellers follow the original factory specifications.

Badly damaged and broken impellers can be repaired by modern welding and flame-spraying techniques. Repairing older, large impellers is generally much less expensive than purchasing new ones.

Electric Motor Maintenance

In keeping with the theme of simplicity as we discuss motor and switchgear maintenance, we will discuss four major considerations which, if adhered to, will eliminate most problems. Almost all electrical failures are a result of neglecting one or more of these considerations.

1. Keep it clean. - Dirt and oil provide an environment for moisture to collect and remain in contact with energized parts, which can greatly reduce resistance. Oil and other contaminants deteriorate insulating materials, as well as provide abrasive materials which will increase wear on moving parts.

2. Keep it tight. - Probably the most common cause of electrical failures is the loose connection. Electric current flowing across a loose connection generates heat causing insulation failure, brittle wires, fires, etc. A regularly scheduled program to inspect and tighten all electrical connections should be followed.

3. Keep it dry. - Water and electrical equipment do not mix. Moisture greatly reduces insulation resistance. An example of this is a 350-horsepower motor that became saturated due to a pump packing failure. Before saturation, the measured insulation resistance was over 1,000 megohms; after saturation, the measured resistance was

less than one-half megohm. This required that the motor be reconditioned before being put back into service. Irrigation sprinkler systems should never be allowed to spray directly onto pump motors and electrical equipment.

4. Keep it cool. - Heat is a major enemy of insulating material especially in older motors and wiring. Studies have shown that electric motor life is doubled for each 18° F reduction in operating temperature. Sunshades for motors; ventilating fans; separate, cooled enclosures for pumping plant switchgear are effective measures to reduce temperatures.

A program of regular insulation resistance testing should be followed. The use of a "Megger" insulation resistance tester will give a good indication of the condition of the entire electrical system. Here again, keep a record of these tests and other maintenance performed. Many potential problems can be detected before they become major failures by taking these tests.

On large motors, one of the best maintenance procedures is a thorough visual inspection. Special attention should be paid to the stator coils. Inspect the condition of the insulating varnish; it should have a good gloss and not be cracked or crazed. Make sure the coil ties at the end turns are tight, and that there is no looseness whatsoever, especially in the area where the coils enter the stator iron. Slot sticks should be inspected; they should be firmly in place. Slip rings and commutators should be inspected. Any that are badly worn should be cleaned and repaired. New brushes should be installed when necessary and properly seated.

When it becomes necessary to regroove the commutator, it can be done commercially on machines specially designed for that purpose. If done by hand in the machine, be certain that the grooves are cut to the proper depth and that the bars are beveled so that none of the insulating mica protrudes to the surface of the bar. Protruding mica will cause uneven wear and arcing during operation.

During the nonirrigation season in harsher climates, motors that are outside should be winterized. Install snowshields that are painted a contrasting color so that the pump will not be operated with them on. Fill the bearing oil reservoirs to submerge the bearings and prevent rusting. Change the bearing oil in the spring or more often if conditions warrant. Use a good grade of oil. Avoid the temptation to economize on oils. Pumps frequently operate continuously for the entire season without changing bearing oil.

Electric motors operating in the typical outdoor environment should be reconditioned on a regular basis. The reconditioning procedure consists of a thorough steam cleaning and ovendrying. The motor stator is then dipped in an insulating varnish and baked.

During the time motors are being reconditioned is an excellent opportunity to inspect and make sure all mechanical parts are in top operating condition. Any problems in the electrical system, such as loose slot sticks, loose end turns or ties, cracked leads, etc., can be repaired or replaced. The rotors should also be carefully inspected. Make sure there are no cracked squirrel cage bars, broken fan blades, or any loose components.

As high as 80 percent to 90 percent of motor failures occur because of a failure of some mechanical component within the pump-motor system. Bearing failure is probably the most frequent. The mechanical failure can then involve the motor windings, leading to a possible complete failure or a very costly rewind job.

The interval between scheduled reconditionings should be determined by the number of motors involved and budgetary considerations. A 5-year interval has proven to be very satisfactory and relatively inexpensive. With this type of program, it has been well documented that failure rates of motors are reduced to less than 2 percent from all causes.

Switchgear Maintenance

Here again, keep in mind the four major considerations outlined earlier, with the emphasis on keeping connections tight and dirt out. In the switchgear as in no other place, the failure of a small, seemingly insignificant component will keep the pump out of service.

The main contactor is the major component of the switchgear. It, along with the rest of the controls, should be inspected and serviced at least annually. Special attention should be given to the contact points. When the contactor closes, the points should meet simultaneously and have equal tension. Any contactor points that are badly pitted or worn should be replaced. Spare contact points and main holding coils should be on hand at all times, for all the makes and sizes of contactors being operated.

Safety

All of the maintenance procedures we have discussed are fairly easy to accomplish while the switchgear is not energized. If you must work it hot, it becomes much more complex. You may get only one mistake. Only highly qualified personnel should attempt to work on energized equipment, and then only in extreme situations.

Transformer Maintenance

The same principles that apply to motor and switchgear maintenance also apply to transformers. One of the best indicators of what is going on inside a distribution transformer is the insulating oil. Test the oil for dielectric strength; this will indicate the presence of water and other contaminants. Testing the oil for acidity is a fairly simple, inexpensive test. This test will indicate when the critical point of acidity is reached and sludge begins to form.

Another problem is that transformer oil may be contaminated with PCB's (polychlorinated biphenyls). This may occur even though the nameplate states that the transformer is filled with mineral oil. Contamination has been known to occur in oil-filled transformers, during manufacture of the transformers, and transportation of insulating oils.

Repair facilities will also require that a test for PCB's be performed before undertaking any repair work.

Conclusion

As stated in the beginning, most of the procedures outlined here are basic and quite simple. Experience has shown that almost without exception, when the cause for a failure has been determined, it can usually be traced to not fulfilling one or more of the procedures discussed here. Almost all electrical-mechanical breakdowns can be prevented by a good, ongoing preventive maintenance program.